

Stormwater and Water Quality Model Users
Group Meeting - Proceedings Held on
January 27-28, 1983

(U.S.) Environmental Research Lab., Athens, GA

Sep 83

U.S. Department of Commerce
National Technical Information Service

NTIS

TECHNICAL REPORT DATA (Please read Instructions on the reverse before completing)		
1. REPORT NO. EPA-600/9-83-015	2.	3. RECIPIENT'S ACCESSION NO. PB84 118454
4. TITLE AND SUBTITLE Proceedings of Stormwater and Water Quality Model Users Group Meeting, January 27-28, 1983	5. REPORT DATE September 1983	
7. AUTHOR(S) Thomas O. Barnwell, Jr., (Ed.)	6. PERFORMING ORGANIZATION CODE	
9. PERFORMING ORGANIZATION NAME AND ADDRESS Environmental Research Laboratory Office of Research and Development U.S. Environmental Protection Agency Athens GA 30613	8. PERFORMING ORGANIZATION REPORT NO.	
12. SPONSORING AGENCY NAME AND ADDRESS Environmental Research Laboratory--Athens GA Office of Research and Development U.S. Environmental Protection Agency Athens GA 30613	10. PROGRAM ELEMENT NO. AARB1A	11. CONTRACT/GRANT NO.
	13. TYPE OF REPORT AND PERIOD COVERED Final	14. SPONSORING AGENCY CODE EPA/600/01
15. SUPPLEMENTARY NOTES		
16. ABSTRACT This report includes 17 papers on topics related to the development and application of computer-based mathematical models for water quality and quantity management presented at the semi-annual meeting of the Joint U.S. Canadian Stormwater and Water Quality Model Users Group held on January 27-28, 1983 in Gainesville, Florida. Topics covered included applications of the QUAL-II model in the Northeastern United States and in Columbia, South America; applications of STORM in Singapore and the Republic of China; and implementation of SWMM in Tunisia. Modeling pesticides in small ponds and mixing zones in rivers also was discussed, as well as sizing detention ponds to meet water quality objectives. A program to assess the impact of forest management practices on water quality was also presented as was a mechanistic model for nonpoint source simulation. Papers related to precipitation discussed data management, estimation of missing values, and areal intensity-duration-frequency curves. The questions of hydrological regionalization, snowmelt, and porous pavements were addressed. Other papers discussed regional stormwater management facilities and storm sewer design optimization.		
17. KEY WORDS AND DOCUMENT ANALYSIS		
a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
18. DISTRIBUTION STATEMENT RELEASE TO PUBLIC	19. SECURITY CLASS (This Report) UNCLASSIFIED	21. NO. OF PAGES 289
	20. SECURITY CLASS (This page) UNCLASSIFIED	22. PRICE

United States
Environmental Protection
Agency

Environmental Research
Laboratory
Athens GA 30613

EPA-600/9-83-015
September 1983

Research and Development

2B84-118454



Proceedings of Stormwater and Water Quality Model Users Group Meeting

January 27-28, 1983

BEST

AVAILABLE

COPY

REPRODUCED BY
NATIONAL TECHNICAL
INFORMATION SERVICE
U.S. DEPARTMENT OF COMMERCE
SPRINGFIELD, VA. 22161

EPA-600/9-83-015
September 1983

PROCEEDINGS
OF
STORMWATER AND WATER QUALITY MODEL
USERS GROUP MEETING
January 27-28, 1983

Edited by

Thomas O. Barnwell, Jr.
Center for Water Quality Modeling
Environmental Research Laboratory
Athens, Ga. 30613

ENVIRONMENTAL RESEARCH LABORATORY
OFFICE OF RESEARCH AND DEVELOPMENT
U. S. ENVIRONMENTAL PROTECTION AGENCY
ATHENS, GA 30613

DISCLAIMER

Mention of trade names or commercial products does not constitute endorsement or recommendation for use by the U. S. Environmental Protection Agency. Similarly, publication of studies reporting better results from one model vis-a-vis others does not constitute endorsement.

FOREWORD

A major function of research and development programs is to effectively and expeditiously transfer technology developed by those programs to the user community. A corollary function is to provide for the continuing exchange of information and ideas between researchers and users, and between the users themselves. The Stormwater and Water Quality Model Users Group, sponsored jointly by the U. S. Environmental Protection Agency and Environment Canada/Ontario Ministry of the Environment, was established to provide such a forum. The group has recently widened its interest to include models other than the Stormwater Management Model and other aspects of modeling water quality in urban and natural waters. This report, a compendium of papers presented at the January 1983 Users Group meeting, is published in the interest of disseminating to a wide audience the work of group members.

William T. Donaldson
Acting Director
Environmental Research Laboratory
Athens, Georgia

ABSTRACT

This report includes 17 papers on topics related to the development and application of computer based mathematical models for water quantity and quality management presented at the semi-annual meeting of the Joint U.S.-Canadian Stormwater and Water Quality Model Users Group held on January 27-28, 1983 in Gainesville, Florida.

Topics covered included an applications of the QUAL-II model in the North-eastern U.S. and Columbia, South America ; applications of STORM in Singapore and the Republic of China; and implementation of SWMM in Tunisia. Modeling pesticides in small ponds and mixing zones in rivers was also discussed, as well as sizing detention ponds to meet water quality objectives. A program to assess the impact of forest management practices on water quality was also presented as was a mechanistic model for nonpoint source simulation. Papers related to precipitation discussed data management, estimation of missing values, and areal intensity-duration-frequency curves. The questions of hydrological regionalization, snowmelt, and porous pavements were addressed. Other papers discussed regional stormwater management facilities and storm sewer design optimization.

CONTENTS

	Page
FOREWORD.....	iii
ABSTRACT.....	iv
Project Success - Sensitivity to Project Assumptions.....	1
Anthony Knepp and Grace Wood	
Dames & Moore	
Application of the STORM Model to Design Problems in Singapore and Kaoh- siung, Republic of China	22
Brendan M. Harley	
Camp Dresser & McKee Inc.	
An Attempt to Implement SWMM in Tunisia.....	43
Janusz Niemczynowicz	
Lund Institute of Technology	
The Sediment-Water Interface in Modeling Pesticides in Sedimentation Ponds	53
Raymond A. Ferrara and Peter Jaffe	
Princeton University	
A Mixing Zone Model for Conservative Parameters	65
Main R. Hutcheson	
Oklahoma Water Resources Board	
Some Recent Adaptations and Applications of QUAL-II in the Northeast	87
William W. Walker, Jr.	
Environmental Engineer	
A Review of Model Use in Evaluating Nonpoint Source Loads from Forest Management Activities	102
George G. Ice and Raymond C. Whittemore	
National Council of the Paper Industry for Air and Stream Improvement	
Detention Pond Sizing to Achieve Water Quality Objectives	123
Roger K Wells	
HMM Associates, Inc.	
A Mechanistic Simulation for Transport of Nonpoint Source Pollutants	146
Daryl B. Simons, Ruh-Ming Li and Kenneth G. Eggert	
Simons, Li & Associates	

Data Management for Continuous Hydrologic Simulation	161
Jy S. Wu	
University of North Carolina at Charlotte	
Estimation of Missing Values in Monthly Rainfall Series	177
Efi Foufoula	
University of Florida	
Areal Intensity-Duration-Frequency Curves - A Possible Way of Improving the Rainfall Input	192
Janusz Niemczynowicz	
Lund Institute of Technology	
Hydrological Regionalization: A Question of Problem and Scale.....	202
I. Simmers and E. Seyhan	
Free University, Amsterdam	
Snowmelt Induced Urban Runoff in Northern Sweden	215
Lars Bengtsson	
McMaster University	
An Advancement in Hydraulic Modeling of Porous Pavement Facilities	237
Gary Goforth	
Espey Huston & Associates Inc.	
Planning and Implementation of Regional Stormwater Management Facilities in Montgomery County, Maryland	255
John M. Crouse(1), Vincent H. Berg(2) and Linda J. D. Mitchell(1)	
(1)Greenhorne & O'Mara and (2)Montgomery County, MD	
Storm Sewer Design Optimization	272
Dong Hoang	
City of Portland	
ATTENDEES.....	281

Project Success: Sensitivity to Project Assumptions
"The Medellin Case"

by
Anthony Knepp, P.E.
and
Grace E. Wood
Dames & Moore, Bethesda, MD

Abstract

The effect of major planning assumptions on the predicted water quality of the Rio Medellin is investigated through application of the Qual II model. The Rio Medellin flows through the center of Medellin, the second largest city in Colombia with a 1980 population of over 1.2 million people. Currently, there is no treatment of municipal or industrial waste. The river receives raw sewage via direct discharge and from numerous polluted tributaries in the developed areas. Nonpoint sources of pollution contribute large quantities of sediment to the river. Practices such as in-stream gravel mining and direct dumping of solid waste add to the large sediment and pollutant loads observed during wet weather periods. All of these factors result not only in very poor biochemical quality of the river, but frequent visual, color, and odor problems. Public health is threatened by the poor quality of the river, and uses are severely limited.

Preliminary results of a major planning study (Greeley and Hansen, CCC, 1982) indicate that a configuration of four treatment plants sited along the modeled portion of the Rio Medellin would cost-effectively meet specified water quality criteria.

This paper presents the results of a sensitivity analysis using Qual II which demonstrates the degree predicted water quality is affected by initial project assumptions, such as:

- o Expected ability to control the nonpoint sources of pollution, including solid waste and land runoff
- o Availability of dilution water
- o Likelihood of meeting wastewater collection goals.

The paper examines the importance of meeting basic assumptions to the attainment of water quality goals. The sensitivity analysis was accomplished by establishing a "baseline" condition and examining the effects on baseline quality of modifying a single assumption or group of assumptions. Changes to the baseline condition included variations in flow, percent wastewater collection, dilution water inflow, nonpoint source inputs, and solid waste contributions.



Study Area Location Map

Introduction

Every water resources study team begins a project with the expectation that it will proceed in a systematic manner from problem identification and collection of pertinent information, through the development of feasible solutions, to the determination of their consequences. Final results usually include the identification of one or, at most, a limited number of feasible solutions. The decision maker(s) are then expected to arrive at a final decision concerning implementation of a selected alternative, and

then proceed to the next problem. Experience suggests that in small projects such a systematic approach works quite well. However, in many larger water resources planning projects, such conceptual neatness is brought closer to reality by the following observations of Loucks (1981):

- o A final solution to a water resources problem rarely exists.
- o Plans and projects are dynamic and evolve over time.
- o The time to study a problem is never adequate to perform "state-of-the-art" mathematical work. The converse of the above statement should also be recognized, i.e., if sufficient time does exist the objectives of the original study will have shifted by the time the study is completed.

Recognizing that these difficulties exist, a major contribution to the planning process is obtained when a sensitivity analysis is conducted for the major project assumptions composing the "Model" used in the analysis of feasible solutions. "Model" is used here to mean the set of formulae coefficients, parameters, etc., which reproduces both prototype behavior and system response to future conditions. In this way, the evolution of project objectives is enhanced and the feasibility of project solutions is recognized early in the planning stages. Most importantly, the objectives, feasibility of alternatives, and the interrelated aspects of the project are brought into focus for the decision makers. Since most water quality planning studies are based on at least a 20 year planning horizon, a knowledge of the impact of major assumptions to the overall success of a project is invaluable to a decision maker as priorities and objectives change from the time of project inception to the end of the planning horizon.

This paper examines the sensitivity of meeting selected water quality criteria to the assumed status of major project assumptions. Emphasis is placed on those project factors which are often classified as assumptions in a study and which, as demonstrated here, can influence the meeting of project objectives.

The background for this paper is a water quality management plan developed for the metropolitan areas in the Aburra Valley surrounding Medellin, Colombia. As part of an overall study of the valley, a river model was calibrated, verified, and used to evaluate potential wastewater treatment plant systems for the area. Aspects of the study included water quality sampling, the estimation of nonpoint pollution, hydrology, sediment, water diversions for dilution, and solid waste studies. The planning horizon for the sensitivity analysis is the year 2000.

This paper is based on results of a prefeasibility study. Simulation results are used to illustrate the importance of carefully performing a sensitivity analysis of major project assumptions with respect to the information the analysis provides to decision makers for the direction of the project. Results and conclusions referenced in this paper may change

considerably during subsequent feasibility and design phases of the project.

Description of Project Area

The Aburra Valley is located in the Andean mountains, 7° north of the equator at an elevation of 1,500 meters (4,920 ft) above mean sea level (MSL). The City of Medellin and nine other municipalities comprise most of the population of the valley. The city of Medellin, the capital of the Department (state) of Antioquia, is located at the center of the valley (Figure 1) approximately 205 km (127 mi) from the Pacific Ocean and 295 km (183 mi) from the Caribbean Sea. It is the second largest city in Colombia with a 1980 population of over 1.2 million inhabitants. Total population in the valley is approximately 2 million.

The climate is mild with monthly average temperatures varying only slightly over the year. The mean temperature is 22.5°C (72.5°F), with temperatures ranging between 13°C (55°F) and 33°C (91°F). Average annual rainfall totals 1,450 millimeters (57 inches). Average annual humidity is 70%. Four seasons are distinguishable from changing rainfall patterns. Two relatively dry seasons occur from December to March and July to August. Two wet seasons occur from April to June and from September to November.

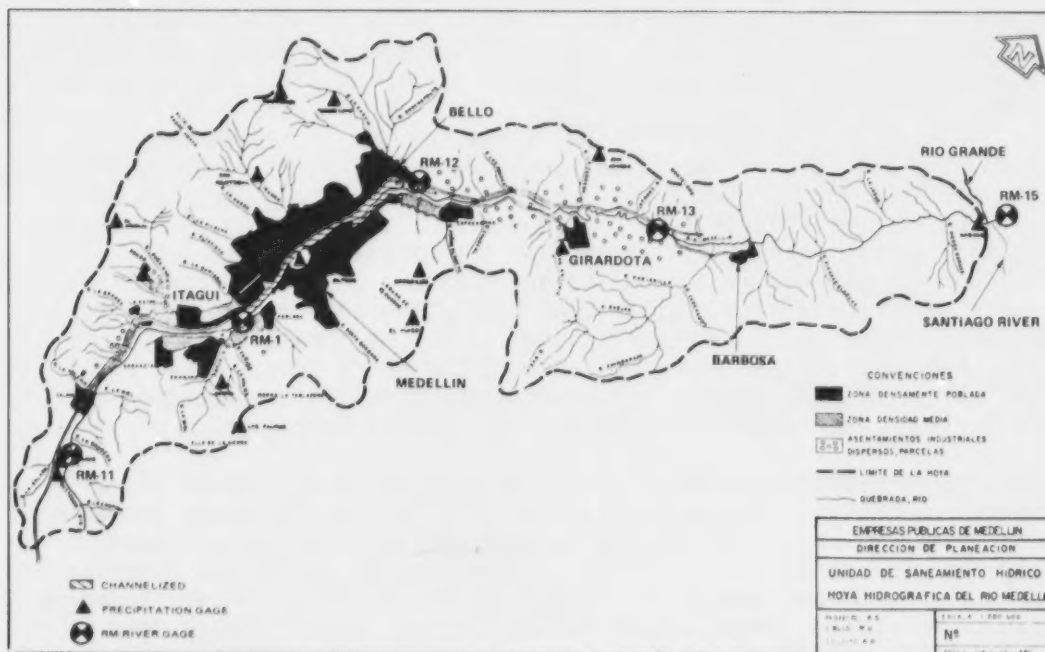


Figure 1: Aburra Valley - study area.

Most of the Aburra Valley is classified as humid subtropical and low mountain humid forest. The forest that remains is concentrated at the upper and lower reaches of the valley. In the Medellin area, urban zones cover most of the available usable land, limited only by steep slopes. The metropolitan area is an important commercial and industrial center for textile activity. The non-urban areas are used for livestock grazing and agriculture.

The Rio Medellin flows north through the urban population centers. The river is the natural drainage for the valley. It receives flow from numerous tributaries, the majority having steep slopes in the upstream sections. Tributary flows range from a few liters per second in the dry seasons to several cubic meters per second during the wet seasons. The Rio Medellin extends for approximately 100 km (62 miles) from its headwater near Primavera to its mouth at the confluence with the Rio Grande. The river receives 7.7 m³/sec (180 MGD) of raw municipal and industrial waste. Median flow at gage RM-12 (Figure 1) is 25 m³/sec (570 MGD). Current uses are severely limited and public health is threatened.

The upper reaches of the Rio Medellin from its headwater to Itagui, approximately 20 km (12 mi), flows through a narrow valley and falls approximately 200 meters (656 ft) in 13 kilometers (8 mi). The water quality in this reach is relatively good. It is downstream of this reach that municipal discharges become significant. Below Itagui the valley widens, the river slope decreases, and the river passes through the heavily populated and industrialized area of metropolitan Medellin. The river is channelized through much of this portion. The water quality in this stretch is severely degraded due to the large volumes of industrial, municipal, and domestic effluents being discharged into the river as shown by high levels of coliform bacteria, oil and grease, detergents, and heavy metals. The five-day biochemical oxygen demand (BOD5) is very high, reaching 120 mg/l in the metropolitan area, and the dissolved oxygen (DO) approaches zero, especially during the dry season. Below the channelized reach, the water quality continues to be highly degraded due to industrial and domestic discharges from Medellin and Bello. As a result, the DO concentration remains near zero. Because of the short travel times and high pollution load the organic matter from the Medellin area decomposes only slightly in the river and water quality remains very poor.

The characteristics of the river below gage RM-12 (kilometer 40, mile 26) changes radically. The valley becomes very narrow, the slope increases, and water flow is very rapid passing through riffle areas and cascades. This stretch represents a zone of high reaeration, as rapids and cascades oxygenate the water and sewage discharges are minimized by the small degree of human development. Thus, oxygen levels increase with distance downstream but BOD5, nutrients, toxic substances, and coliform bacteria concentrations are still elevated.

Near the mouth of the Rio Medellin, the valley widens and the slope again decreases. The Rio Grande and the Rio Santiago join the Medellin to form the Rio Porce and dilute the pollutants of the Rio Medellin.

Figures 2 through 5 present water quality measurements taken between March and August 1981 (Dames & Moore, 1981). Flows were generally above the average annual levels. Sampling locations are indicated by the vertical lines and ranges of measurements by short dashes. The sampling points are connected for clarity.

Figures 2 and 3 indicate the extensive pollution load the river is currently receiving. Oxygen levels are recorded below 2 mg/l over a 20-km (12 mi) reach of the river. BOD5 concentrations are greater than secondary effluent levels (30 mg/l) for nearly 40 kilometers (24 mi). Ammonia nitrogen and total phosphorus concentrations are well above potential eutrophication causing levels.

Waste Management Practices in the Aburra Valley

The city of Medellin and the surrounding municipalities are without facilities necessary for the treatment of domestic, industrial, or commercial wastewater. Although the metropolitan area is served by sanitary sewers, these discharge directly to the mainstem or to various tributaries and open ditches draining the area.

Existing management practices for solid waste disposal also contribute to the poor quality of the Rio Medellin. The indiscriminate disposal of solid waste directly into and on the banks of the river and its tributaries creates obvious visual and odor problems. The organic load from floating waste and the accumulation of debris creates a potential demand on in-stream oxygen and elevates the concentration of BOD5, thereby adding to the degradation of the river.

Water Quality Criteria

Quality criteria were established for a range of potential uses of water from the Rio Medellin. Criteria were developed for two categories of pollutants: pollutants not significantly affected by the abatement program, pollutants significantly affected by the abatement of organic pollution. The organic pollution criteria served a dual purpose. First, they obviously indicated a concentration of the given pollutant. Second (and more important) they served to indicate a level (albeit subjective) of chemical pollution, originating from industrial and commercial activities in the valley. The implementation program is expected to alleviate increasingly high percentages of pollution from the above two sources as higher percentages of the population related pollution is eliminated.

Water quality parameters included as indicators of organic pollution were dissolved oxygen, biochemical oxygen demand, ammonia, total phosphorus, and fecal coliforms.

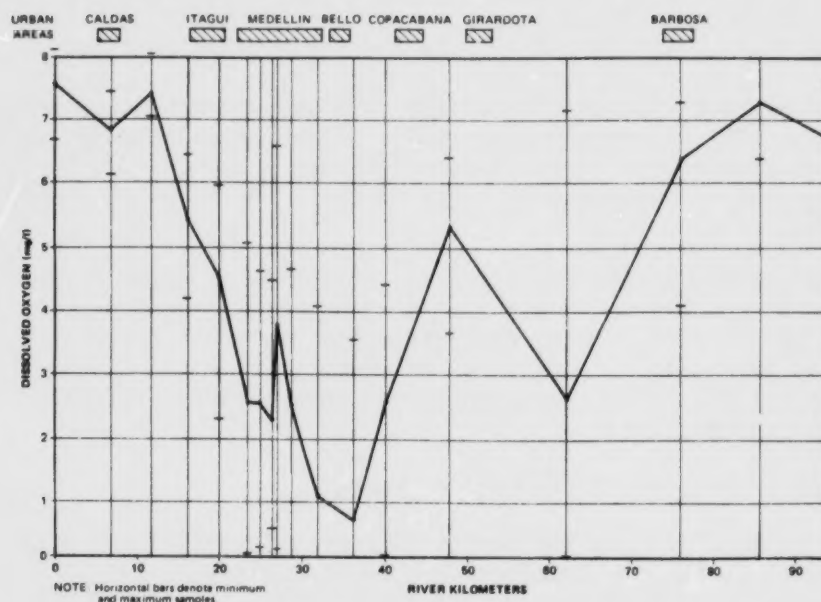


Figure 2: Rio Medellin mean dissolved oxygen for March - August, 1981.

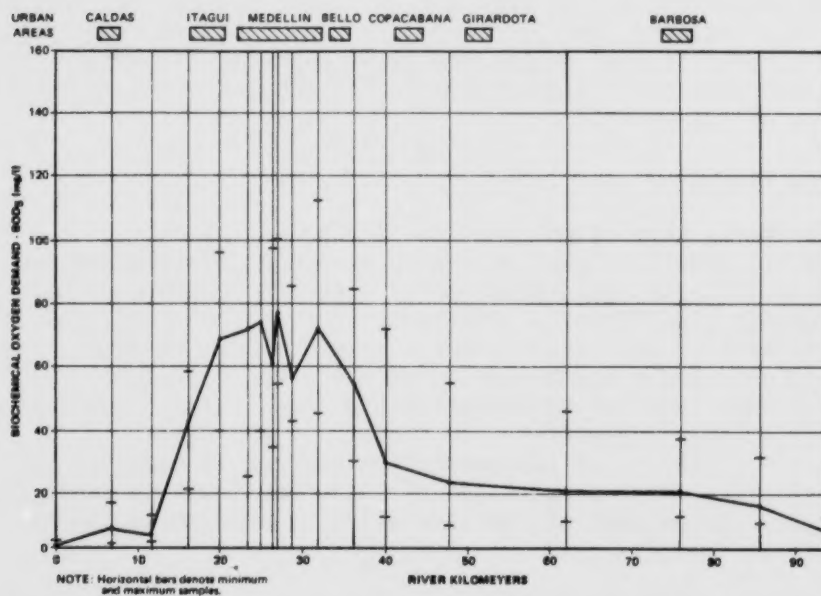


Figure 3: Rio Medellin mean biochemical oxygen demand for March - August, 1981.

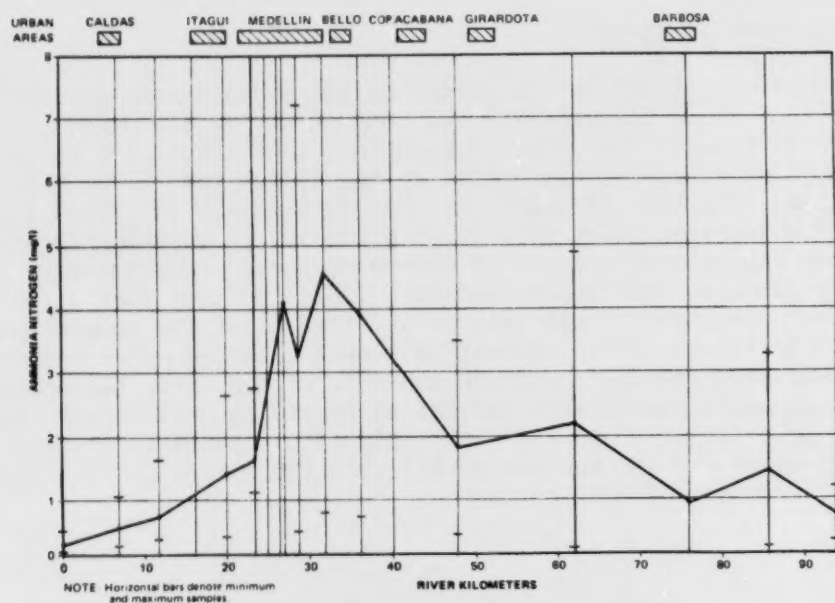


Figure 4: Rio Medellin mean ammonia nitrogen for March - August, 1981.

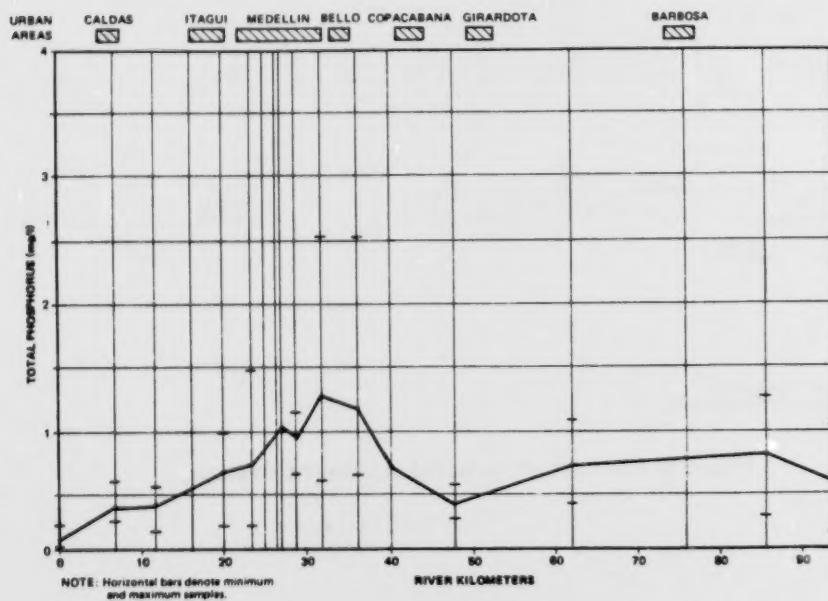
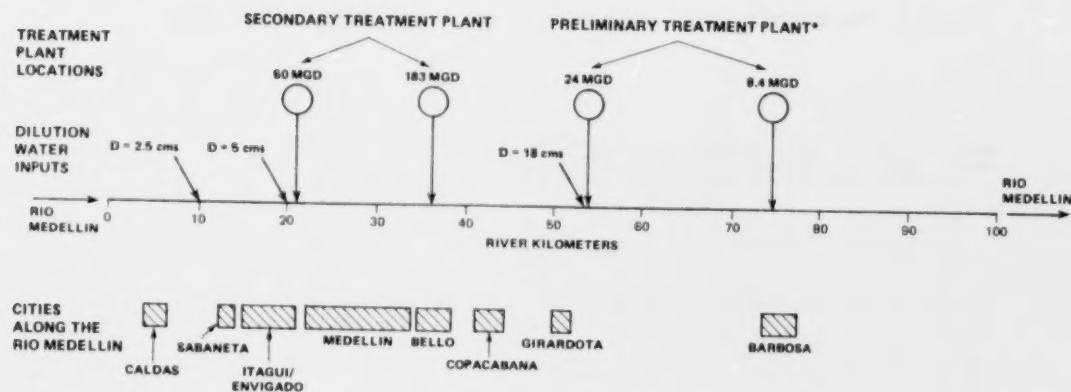


Figure 5: Rio Medellin mean total phosphorus for March - August, 1981.

Selected Abatement Strategy

An extensive analysis was performed by the project team (Greeley and Hansen, and CCC, 1982) to determine the cost-effective abatement strategy appropriate for the Rio Medellin. Population, land use, solid waste, and river mining conditions were projected to the design year 2000 to estimate pollutant loads. The Qual II model was used to simulate in-stream quality resulting from implementation of a given point source treatment plant configuration and selected non-point source controls. Preliminary results of the study (Greeley and Hansen, and CCC, 1982) indicate that a configuration of four treatment plants (Figure 6) sited along the modeled portion of the Rio Medellin would cost-effectively meet selected water quality criteria. The proposed configuration consists of two large secondary plants and two additional plants defined as providing preliminary treatment (screening, grit removal, and chlorination). The interbasin transfer of high quality water for the purpose of dilution ($20.5 \text{ m}^3/\text{sec}$) is also assumed in the treatment configuration.



*PRELIMINARY TREATMENT IS DEFINED AS
SCREENING, GRIT REMOVAL AND CHLORINATION

Figure 6: Point source selected abatement strategy.

Sensitivity Analysis

Adapting the calibrated model to year 2000 conditions required many assumptions and projections to be made which describe the expected "state" of the environment. Foremost among these were the assumptions regarding:

1. Level of naturally occurring base flow used in the screening of alternatives.
2. Year 2000 status of the interceptor connection program to the major trunk lines.
3. Availability of an appreciable quantity of dilution water, particularly during drought periods when dilution water is most needed.
4. Level of solid waste control possible.
5. Effects of wet weather (nonpoint) sources of pollution on meeting quality criteria.

The implementation of a comprehensive plan requires a detailed knowledge of how the quality of the river responds to each of these assumptions and which among them are impediments to meeting water quality criteria. The potential range of changes in regard to political and social forces that are possible over a 20-year planning horizon make an understanding of the sensitivity of the proposed solution to these assumptions critical during the implementation phase of the comprehensive plan.

Sensitivities were developed by modifying either a single assumption or group of assumptions and using the Qual II model to develop the associated water quality response (Dames & Moore, 1982e). A "baseline" simulation was used as a standard to compare the relative sensitivities. The "baseline" simulation represented a most likely set of assumptions regarding the development of events influencing water quality. For the baseline simulation, a treatment plant configuration of four plants was selected--two secondary plants located at Envigado and Bello, and two plants providing preliminary treatment at Girardota and Barbosa. The two secondary plants were assumed to have 92% treatment efficiency for BOD removal.

The status of the five major assumptions in the Baseline simulation are:

1. A flow likely to be exceeded 75 percent of the time was selected as the naturally occurring portion of the flow used in the baseline simulation.

2. Seventy-five percent of the domestic, industrial, and commercial wastewater in the valley is collected and treated, with 95% collection and treatment of industrial waste from several major industries in the valley.
3. High quality dilution water is available to the Rio Medellin.
4. Implementation of a solid waste management program eliminates the pollutant loadings attributed to unmitigated solid waste dumping.
5. Nonpoint sources of pollution are minimal under baseline conditions.

In the following discussion each project assumption is examined separately. The basis for each is outlined and the project details surrounding the assumption are discussed, along with the sensitivity of predicted water quality to a modification of the assumption.

Assumption 1: A flow that is likely to be exceeded 75 percent of the time was selected as the naturally occurring portion of the flow used in the baseline simulation.

The selection of a design flow for use with the Qual II model in the evaluation of the water quality impacts of abatement programs is a major factor basic to the results of much of the analysis. Currently, no local institutional or legal constraints exist in Colombia to guide the selection of this flow.

An analysis of extreme events, the usual procedure for determining a design low flow, was not possible due to the short length of records available. The period of record was 2-10 years, with an average of 6 years at each gage (Dames & Moore, 1982c). Therefore, for purposes of screening abatement strategies a flow exceeded 75 percent of the time was established for the analysis. The exceedance frequency was selected in recognition of the severity of existing conditions and uncertainties in the data.

To demonstrate the sensitivity of this assumption on predicted water quality, the Qual II inputs were modified to reflect a more severe flow condition, the average annual 14-consecutive-day low flow. The effect of modifying the design is shown in Figure 7. The reduced flow corresponds to an exceedance frequency of 90 percent. The minimal impact on oxygen reserves of a lower flow is due to the large quantities of dilution water and sanitary contributions to the total flow in comparison to the smaller "naturally" occurring base flow portion. The predicted maximum DO sag level is less than 0.5 mg/l more severe. The predicted in-stream BOD5

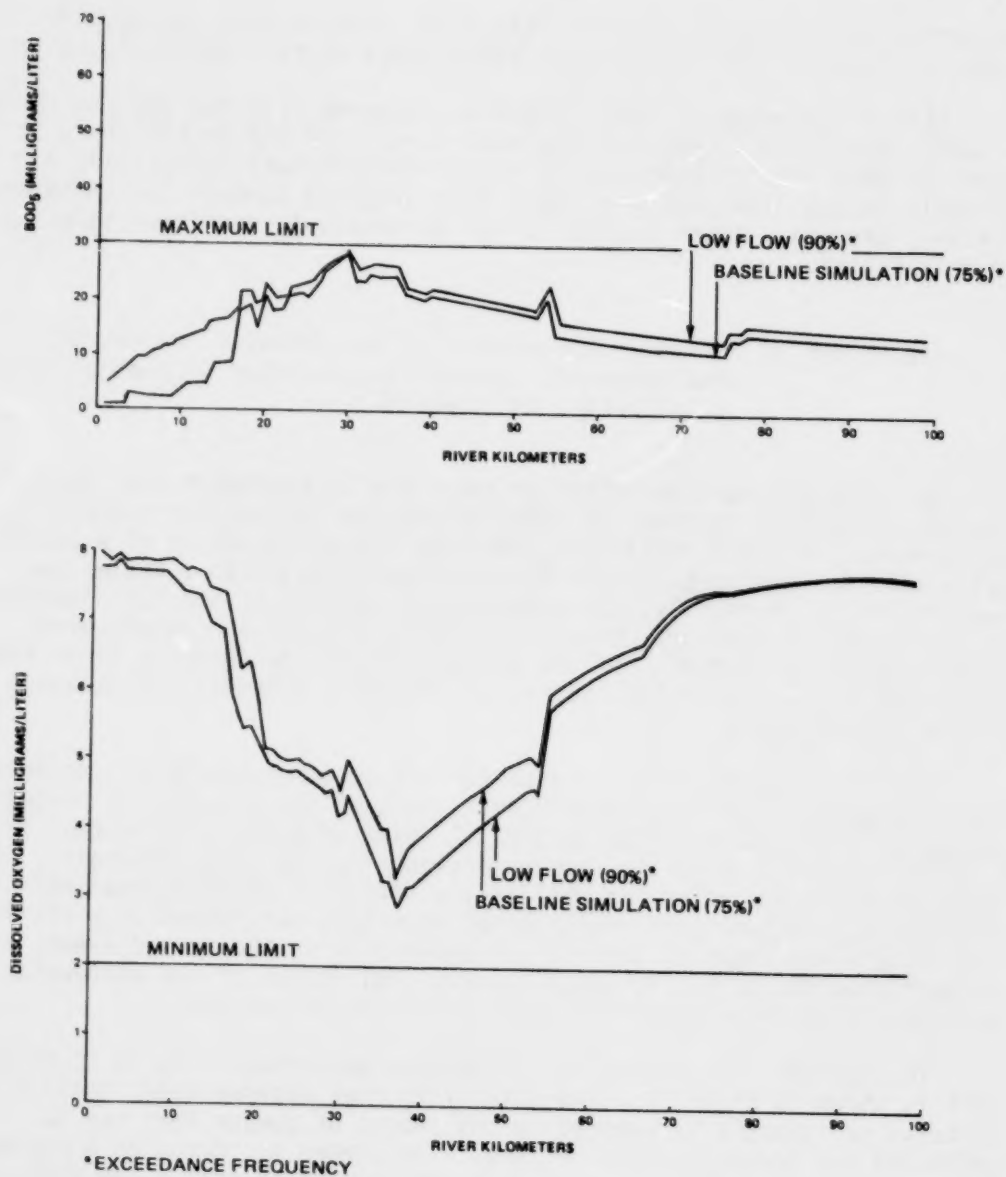


Figure 7: Effect of changes in flow on biochemical oxygen demand and dissolved oxygen concentrations.

concentrations are only minimally affected. The maximum and minimum limits shown on Figure 7 are those limits established to minimize odor problems.

With a knowledge of local hydrology, coupled with the low sensitivity of water quality to changes in the base portion of the design flow, it becomes evident that further work in establishing a more restrictive or reliable design flow should not be a high priority concern in the future planning process. Water quality is not particularly sensitive to changes in this factor.

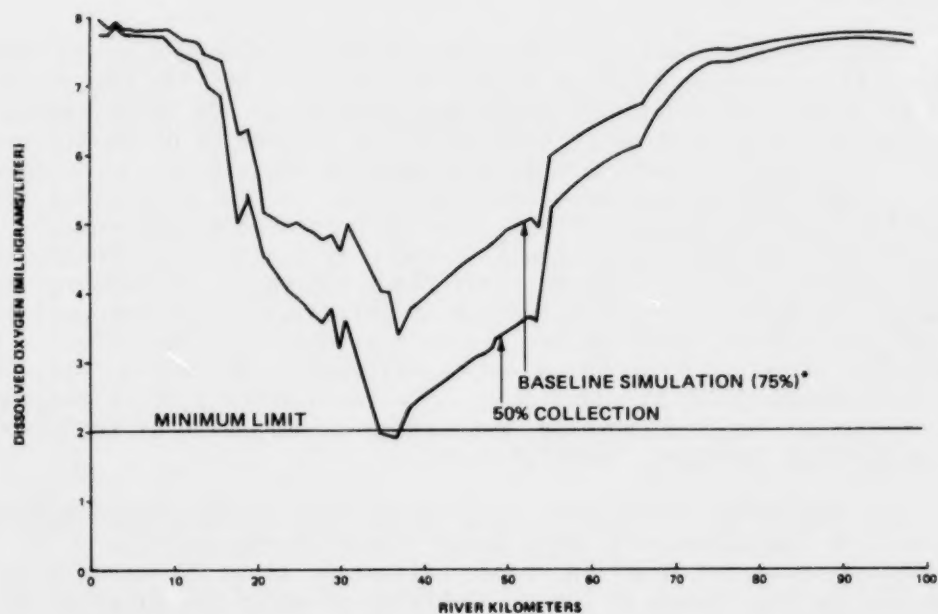
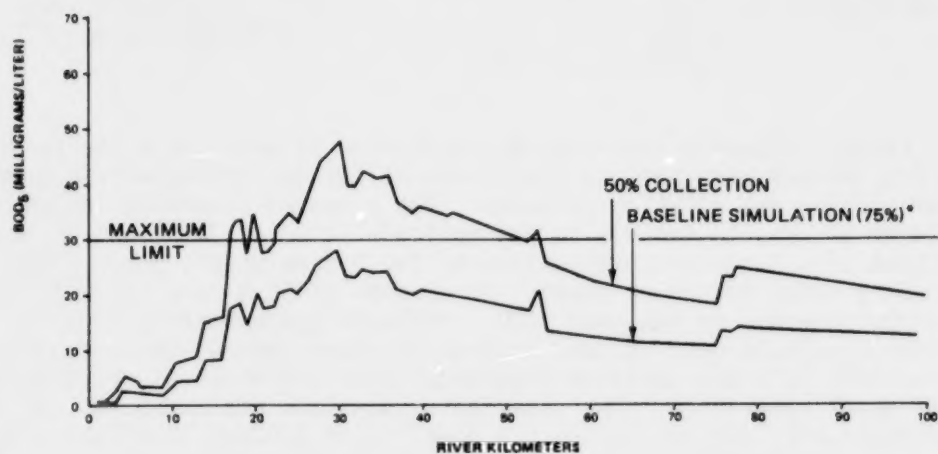
Assumption 2: Seventy-five percent of the domestic, industrial and commercial wastewater generated in the valley is collected and treated.

As indicated earlier, although there are no treatment facilities in the Aburra Valley, portions of Medellin and the surrounding municipalities are sewered. In 1957 a plan was completed for construction of a sanitary collection and treatment system for the City of Medellin (Greeley and Hansen, 1957). Presently, approximately 50 percent of the 152 kilometers of the major interceptors (94 mi) originally planned are constructed. A major problem is that many of the trunks and smaller domestic lines are not connected to the main interceptors and necessary manholes and cleanout points are not built.

A large-scale retrofit program for the system connections, manholes, overflow outlets, etc., is necessary as part of the plan for 75 percent wastewater collection in the valley. A point of concern is whether 75 percent collection is feasible from an engineering and cost standpoint. The topography of the region, as well as the status of existing housing conditions, precludes collection of an uncertain percentage of waste flows. Many of the dwellings in the valley, especially in the upland areas, are without plumbing or public water supply. Collection of the wastewater generated from these houses would be difficult to achieve.

The results of changing the collection percentage from 75 to 50 percent is shown in Figure 8. The ability to meet minimum water quality criteria is strongly influenced by the amount of sewage that can be collected and treated before release. The oxygen sag drops to a minimum 2.0 mg/l while the BOD5 increases to a maximum of 48 mg/l. The BOD5 maximum and DO minimum occur near the center of the most populated area.

A major public works program to connect 75 percent of the wastewater flows to the collection/treatment system must be recognized as a critical factor in achieving water quality goals. The indicated sensitivity of the predicted water quality would lead a decision maker to recognize that the program to connect interceptors to the main trunk lines is a major factor in the implementation of a comprehensive plan.



*WASTEWATER COLLECTION

Figure 8: Effect of wastewater collection levels on biochemical oxygen demand and dissolved oxygen concentrations.

Assumption 3: High quality water is available for dilution of waste flows entering the Rio Medellin during critical periods.

Current planning calls for the diversion of water from the neighboring Rio Grande watershed to the Aburra Valley for hydroelectric power, water supply, and dilution purposes. The proposed diversion includes the building of gravity tunnels to the municipalities of Bello and Girardota. The plan also includes construction of two hydroelectric power plants as well as a water treatment plant. An average of $35 \text{ m}^3/\text{sec}$ ($1236 \text{ ft}^3/\text{sec}$) would be diverted by the year 2000. Although approximately $12 \text{ m}^3/\text{sec}$ ($424 \text{ ft}^3/\text{sec}$) would be used to meet increasing water demand, the remaining $23 \text{ m}^3/\text{sec}$ ($812 \text{ ft}^3/\text{sec}$) would be discharged into the river as dilution water after power generation. The baseline simulations accounted for the above by establishing inflows to the Rio Medellin of 5.0 and $18 \text{ m}^3/\text{sec}$ (176.6 and $636 \text{ ft}^3/\text{sec}$) at river kilometers 30 and 54 (miles 19 and 34), respectively. An additional $2.5 \text{ m}^3/\text{sec}$ ($88 \text{ ft}^3/\text{sec}$) will be available from another source at kilometer 20 (mile 12).

Since the availability of dilution water is dependent upon completion of the rather complex proposed diversion plan, the ability to meet the initial project assumption is again important to the decision makers. To test the sensitivity of water quality to the assumption of no dilution water, all dilution flows were eliminated from the simulation. The results of this modification are shown for BOD5 and DO in Figure 9, along with the predicted baseline concentrations. The BOD5 concentrations remain elevated through the City of Medellin and only begin to decline downstream as the organic load is diluted, river flows increase, and BOD5 inputs are reduced. Upstream of river kilometer 20 (mile 12), predicted water quality is the same for the baseline and modified assumptions simulations. The simulation of DO is shown to be very sensitive to the availability of dilution water. Without added dilution water a major zone of anaerobic conditions is likely to develop, even after construction of the assumed collection and treatment facilities.

The importance of dilution water to meeting project water quality criteria is indicated to be very large, particularly near the population center of Medellin, river kilometers 15 to 35 (miles 9 to 21). Likewise the importance of the availability of water for dilution to the success of the project is also large.

Assumption 4: A solid waste management plan is assumed to be implemented thereby eliminating solid waste as a cause of water quality degradation.

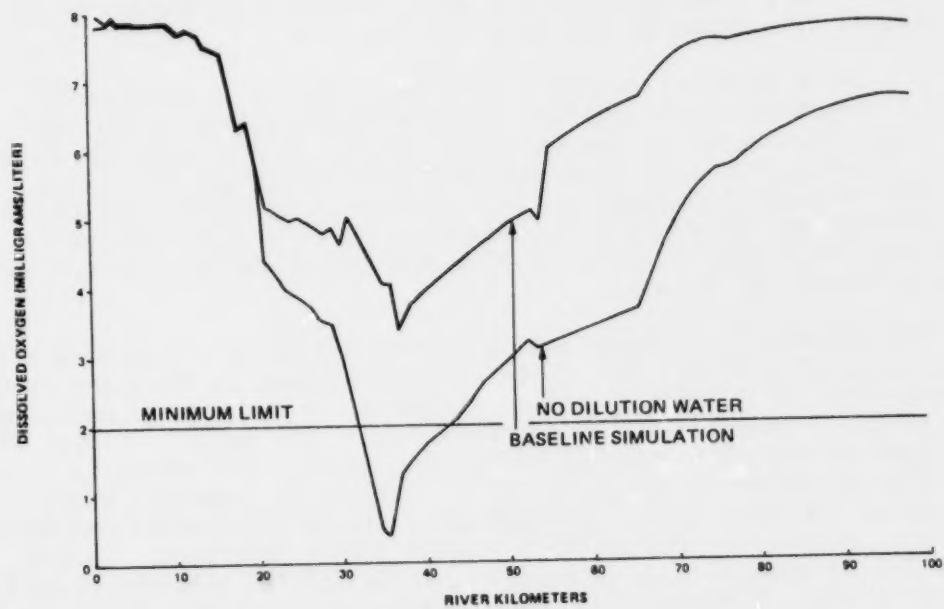
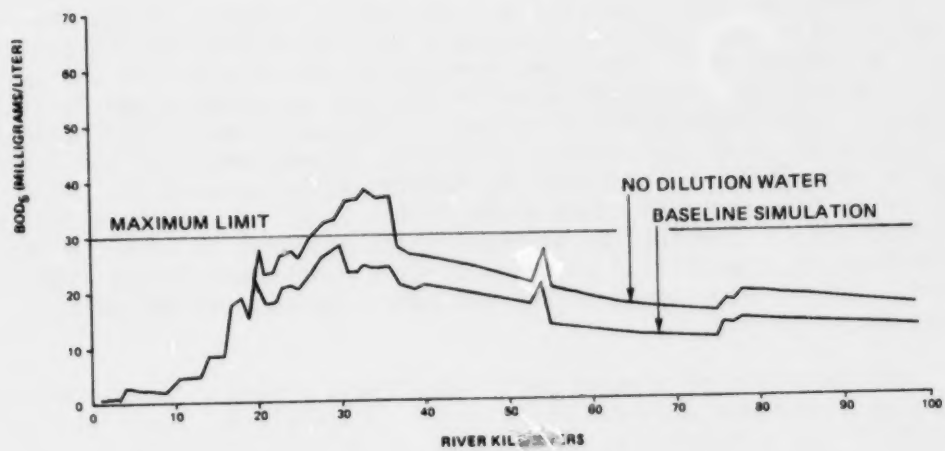


Figure 9: Effect of dilution water on biochemical oxygen demand and dissolved oxygen concentrations.

In 1980, the total quantity of solid waste produced in the Aburra Valley was estimated to be approximately 460,000 metric tons (Grandjean, 1980). The collection, treatment, recycling, and disposal of this solid waste is a complex problem in Medellin and the surrounding municipalities. Although a significant portion eventually reaches a controlled landfill or the composting plant for ultimate disposal, in many areas of the valley the location and methods of disposal are determined by convenience. A common practice is to dispose of solid waste on the banks of the Rio Medellin or its tributaries. Eventually, disposal sites adjacent to water courses extend into the channel and waste is carried downstream during high flow periods, causing widespread visual and odor problems and degrading in-stream water quality.

Implementation of a solid waste management program, independent of a wastewater abatement program, is intended for Medellin. For planning purposes it was assumed that implementation of such a program would eliminate solid waste as a cause of water quality degradation. Therefore, the estimated contribution of solid waste to the total pollution load to the river from all sources (Dames & Moore, 1982b) was excluded as input to the QUAL II simulation for the baseline condition.

The above decision assumes a significant change in solid waste disposal practices in the valley. Solid waste recycling at all levels, at the street receptacle, on the collection trucks, and at numerous disposal sites, provides income to thousands of people throughout the valley. Changing methods of disposal may have serious socioeconomic impacts on many people, and therefore implementation of a program to completely eliminate solid waste as a source of water quality degradation may be quite difficult to achieve.

The sensitivity of water quality to the assumptions regarding solid waste is shown in Figure 10. Solid waste is assumed to enter the river as leachate from solid waste sites (a relatively minor source of pollution), through direct dumping, and by entrainment with the runoff. Figure 10 was developed assuming a storm flow condition existed. The naturally occurring base flow of the river was increased to simulate an "average" storm. It is recognized that Qual II is a hydraulically steady state model and does not simulate the time varying nature of runoff from a watershed. However, the model does provide a crude screening suitable for the sensitivity analysis presented here. Elevated BOD5 concentrations are at almost twice the baseline concentration along the entire modeled portion of the river. The dissolved oxygen sag is approximately 1 mg/l more severe at the point of minimum DO concentration. Recovery of oxygen to concentrations similar to the baseline condition occurs downstream of kilometer 51, (mile 32) because of the addition of 18 m³/sec (636 ft³/sec) of water for dilution.

Meeting water quality criteria is shown to be very dependent on an effective solid waste management program. The reliance of the predicted pollution abatement to the assumption regarding the solid waste management

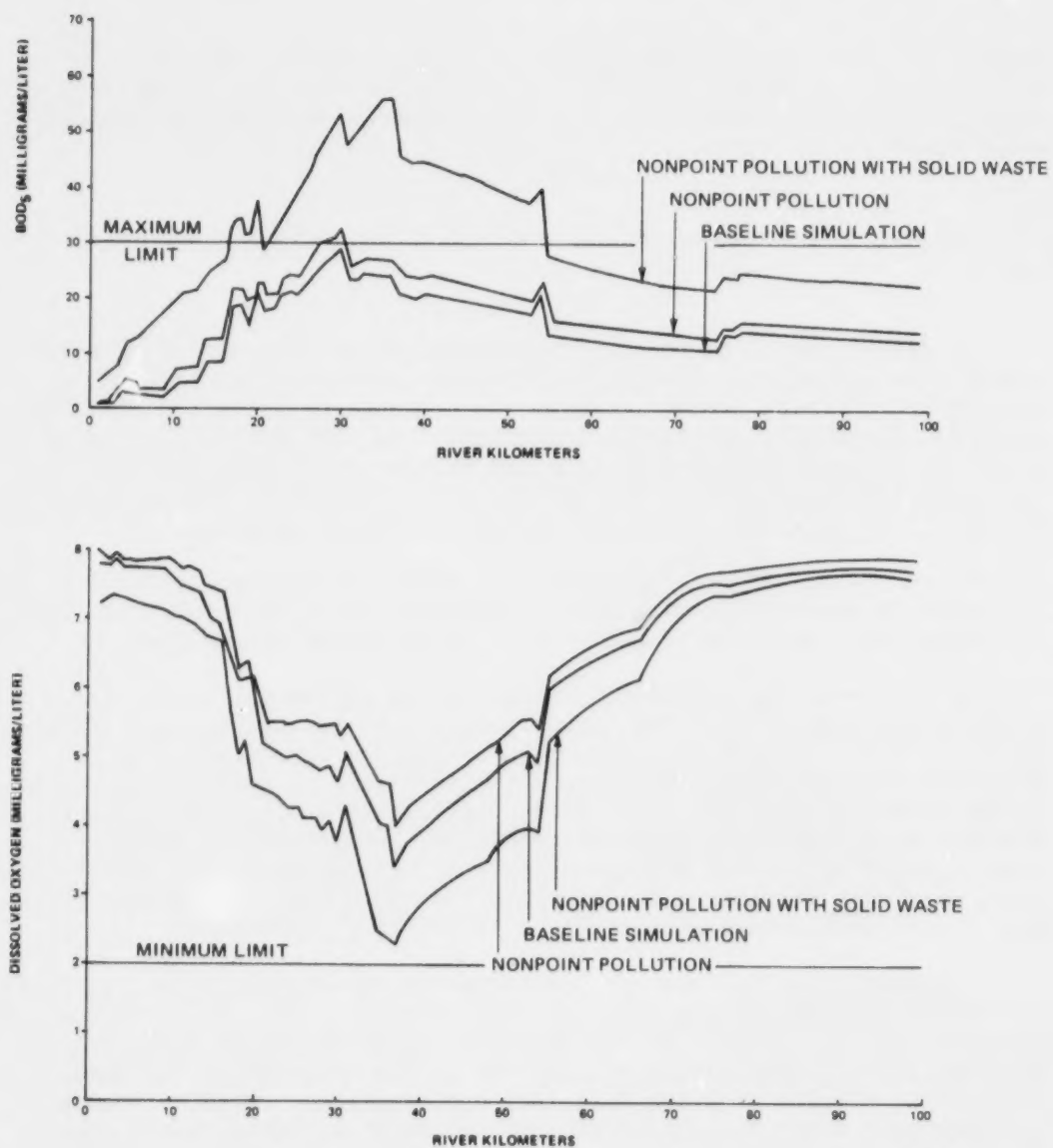


Figure 10: Effect of nonpoint sources and solid waste on biochemical oxygen demand and dissolved oxygen concentrations.

program is clearly indicated, particularly with respect to organic pollutant concentrations. Future implementation of a comprehensive plan should also rank a solid waste management plan as necessary to achieve water quality improvements.

Assumption 5: The effect of nonpoint source pollution on water quality.

By definition here, nonpoint pollution occurs only during rainstorm events when accumulated pollutants on urban and rural land surfaces are carried to receiving streams. Hence, this source of pollution is largely dependent on rainfall patterns, topography, and land use in the region. Since the selected design flow is a dry weather condition, nonpoint pollutant loadings would be minimal and were, therefore, excluded as input to Qual II in selection of an appropriate abatement program.

An important concern to planners, however, is whether nonpoint pollution in combination with point discharges would be significant enough that water quality criteria would be violated during wet weather.

To determine the nonpoint contribution to stream pollution, an estimate was made of diffused, runoff-associated pollutant loads (Dames & Moore, 1982d). Figure 10 presents the results of the simulation. Simulated BOD5 concentrations were slightly higher in wet weather relative to the baseline condition. Below river kilometer 40 (mile 24), concentrations were similar to baseline conditions because of the large quantities of water available for dilution. Dissolved oxygen levels, however, were high throughout the stream. The sag during wet weather was more than 1 mg/l less severe when compared to baseline conditions.

Interestingly, wet weather conditions do not seem to affect seriously the opportunity to achieve water quality criteria. This is in part due to the level of the criteria as indicated on Figure 10 and to the averaging of flows and pollutant loadings required by the methodology followed. First flush effects of urban areas are not simulated. Results indicate that for most water quality parameters, nonpoint pollution would have little impact on achieving water quality criteria at the levels indicated.

Assumption 6: Cumulative Impacts

In the above cases, changes to the baseline conditions included variations in flow, percent collection, water for dilution, nonpoint source pollutants, and solid waste pollutants. In a subsequent step, the cumulative effect of the assumptions impacting water quality was also evaluated by modifying several assumptions simultaneously. These changes

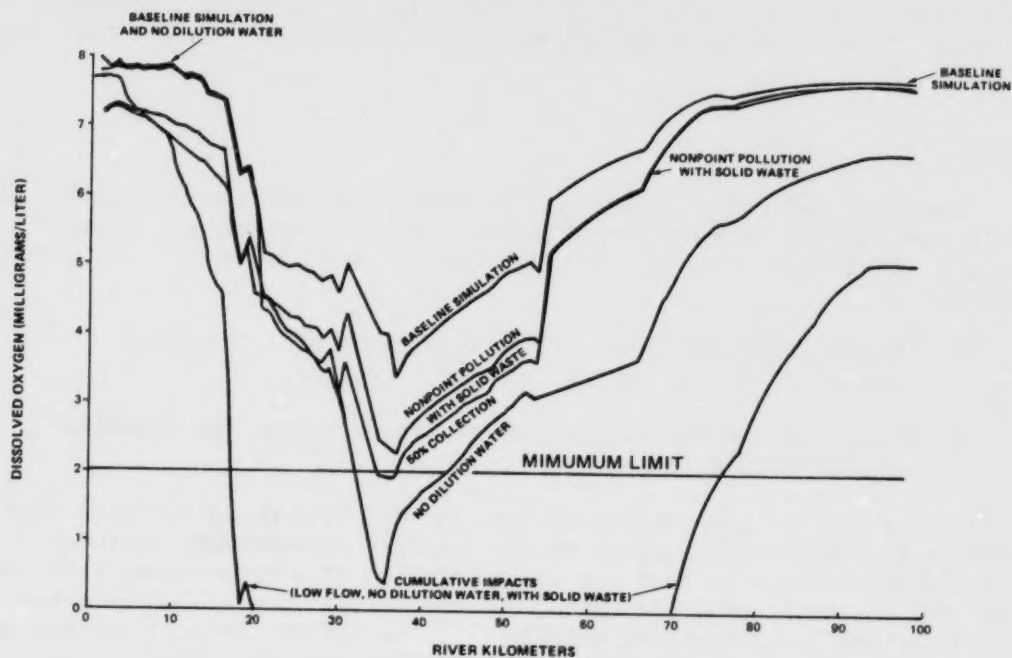
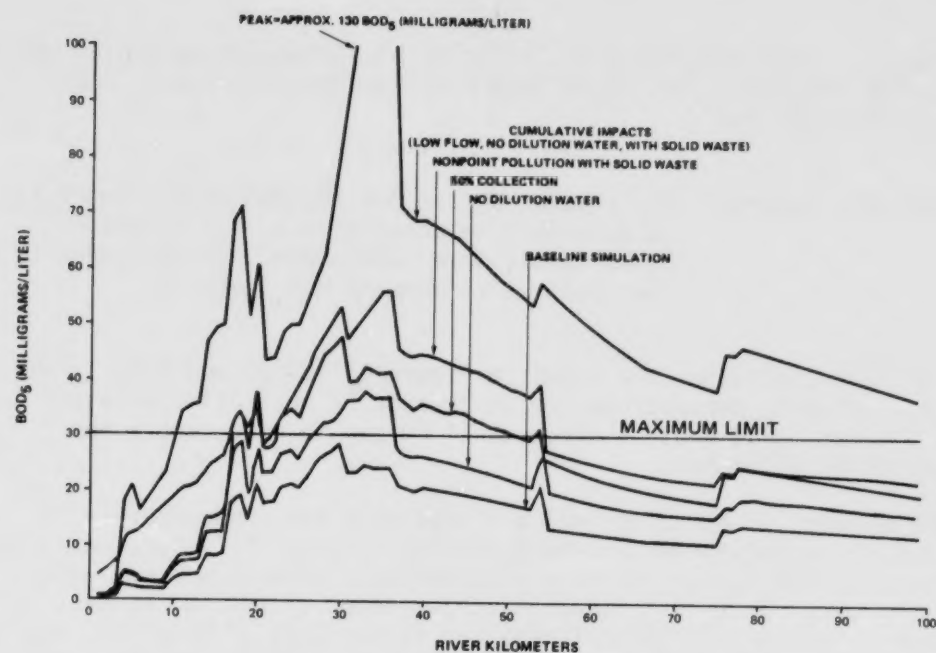


Figure 11: Effects of cumulative impacts on biochemical oxygen demand and dissolved oxygen concentrations.

were chosen to reflect a potential "worst case" picture of resulting water quality. The simulation was based on the following assumptions, modified from baseline conditions:

Cumulative Impacts: The simulated flow was changed to the lower flow as described in Assumption 1. All dilution water inputs were eliminated. No abatement to the solid waste dumping was included.

Results of this simulation are shown in Figure 11 for DO and BOD5. The results more clearly demonstrate the importance of both dilution water availability and solid waste abatement. A DO sag to zero occurs for approximately 50 kilometers.

BOD5 demands exceeding 30 mg/l are predicted for approximately 88 kilometers of the 100-kilometer Rio Medellin. Thirty milligrams per liter of BOD5 was the maximum permissible concentration to meet minimum water quality criteria. This concentration was exceeded except in the most upstream reach. A peak of 130 mg/l resulted in the City of Medellin, where organic loading contributions to the river are the greatest. These results again illustrate the importance of recognizing the relative importance of the major assumptions to achieving water quality planning objectives.

Summary

When used correctly, water quality models can enhance the likelihood of the success of comprehensive planning efforts. The use of a model to evaluate the effect on predicted water quality of major project assumptions provides:

- 1) Understanding of the relative importance of each project assumption to the overall abatement program.
- 2) Basis for informed discussion on the direction the planning process should follow.

An analysis of project assumptions for Medellin would indicate that in addition to the extensive point source controls recommended, decision makers should recognize that the implementation of a sewer connection program, the abatement of solid waste, and the provision of dilution water are very important to maintaining quality criteria during critical periods and thus to the success of the project as currently planned. Also, in this case, the sensitivity of projected water quality to nonpoint sources and to lower flows than used in the screening of abatement alternatives is small.

Acknowledgements

The number of people involved in this study and their valuable contributions are simply too numerous to mention here. Their contributions to the analysis presented in this paper are gratefully acknowledged. Project directors from the organizations involved in the study were Messrs. Robert Zimmerman from Greeley and Hansen, Jaime Rodas from Compania Colombiana de Consultores, Richard C. Tucker from Dames & Moore, and Dr. Alonso Palacios from Empresas Publicas de Medellin. The direction provided the project by their participation shaped the project and helped it successfully to meet its objectives.

REFERENCES

- Dames & Moore, December 1981, "Existing Environmental Conditions," Pollution Control of the Medellin River and its Tributaries - Memorandum 34-1.
- Dames & Moore, April 1982. "Solid Waste Pollution Assessment," Pollution Control of the Medellin River and its Tributaries - Memorandum 56-1.
- Dames & Moore, May 1982. "Hydrologic Studies," Pollution Control of the Medellin River and its Tributaries - Memorandum 59-1.
- Dames & Moore, May 1982. "Nonpoint Pollution Assessment," Pollution Control of the Medellin River and its Tributaries - Memorandum 25-1.
- Dames & Moore, July 1982. "Develop Initial Treatment Arrays," Pollution Control of the Medellin River and its Tributaries - Memorandum 22-1.
- EPA Environmental Research Laboratory, 1977. User's Manual for Stream Quality Model (Qual II).
- EPA, Environmental Research Laboratory, 1977. Computer Program Documentation for Stream Quality Model (Qual II).
- Grandjean, O, 1980, Estudio Sobre Recoleccion, Tratamientos y Disposicion Final de Desechos Solidos en el Area Metropolitana y Oriente Crecano, Parts 1 y 2. Departamento Administrativo de Planeacion y Direccion Desarrollo Metropolitano, Gobernacion de Antioquia.
- Greeley and Hansen, 1957. "Informe Sobre Recoleccion y Disposicion de las Aguas Negras de Medellin. Medellin, EE. PP. M.
- Greeley and Hansen and Compania, Colombiana de Consultores, Ltda., July 1982. "Initial River Model and Treatment Model Analysis of Treatment Configurations and Selection of Treatment Configurations for Further Study," Pollution Control of the Medellin River and its Tributaries, Memorandum 28-1.
- Loucks, Daniel P. et al. Water Resources System Planning and Analysis, Prentice Hall, 1981.
- Orozco, A., 1978. Solid Waste in Medellin and its Metropolitan Area.
- Revista Empresas Publicas de Medellin, Monografia Del Rio Medellin. Vol. 3, Nos. 3 y 4, Julio/Diciembre 1981. ISSN 0120-1239.

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

Application of the STORM Model to Design Problems in Singapore and in Kaohsiung, Republic of China

by

Brendan M. Harley

Camp Dresser & McKee Inc.

Boston, Massachusetts 02108

ABSTRACT

This paper presents two examples of the application of modified versions of the STORM model to design situations overseas. One of these, involved simulation of a raw water supply system where flow is captured via diversions from the storm drainage system. The second application used the model to evaluate cost-effective CSO control measures in a situation where no separation of sanitary and storm sewers exist.

INTRODUCTION

Problems in managing urban runoff and combined sewer overflow in rapidly expanding cities in developing countries are generally quite major and, in some cases, address urgent public health hazards with immediate needs for solutions. Use of simulation models to assist in developing design criteria is still quite rare, although a trend towards use of such models is developing.

Recently, CDM has been involved in two overseas projects which presented the opportunity to apply modified versions of the STORM model to quite different situations. The first, in Singapore, involved a study where urban stormwater runoff from "new town" areas is to be captured and treated for use to augment the island's water supply. The STORM model was used to evaluate the basin runoff, to size the diversion/storage/pumping facilities which are spread throughout the area, and to help determine the effective "safe yield" from the proposed project. This project is now under construction.

The second application of the STORM model was in the development of design strategies to control heavily contaminated combined sewer overflows to the Jen Ai River in the City of Kaohsiung, Republic of China. This city of about one million people has essentially no sanitary sewerage system at all; all sanitary wastewater together with stormwater flows via small open ditches and channels to the river which is presently totally anoxic. The design study was for a series of major combined sewer overflow structures which, in con-

junction with a single trunk sewer, will reduce BOD loadings on the Jen Ai River by over 90 percent. Effective use was made of the ability of the STORM model to simulate the effect of storage and local treatment (simple detention, swirl separators, etc.) at each of the control facilities.

SELETAR PROJECT - SINGAPORE

BACKGROUND

The expansion of reliable water supply systems is a major requirement in areas of the world which experience very rapid population growth. Some of these developing countries lie in regions of the world having limited rainfall, many lie in areas of abundant rainfall but possess very limited capture/storage/transmission capability; in nearly all cases, large capital investments have to be undertaken to meet the water supply needs.

The island nation of Singapore is handling the population growth demands through the development of self-contained "new towns" containing both residential and industrial areas; these "new towns" incorporate separate storm and sewer systems designed to good modern practice levels. Each "new town" is, in effect, a small city with a population of 200,000-400,000. This scheme utilizes runoff from such "new towns" areas to augment the water supply requirements. The scheme which is presently in the final design and tender stages involves the following elements:

- o Diversions from main storm drains to a series of storage ponds located throughout the "new towns." Diversion structures include both large drop-inlets and tightly controlled bascule-gate regulators.
- o Pumpage from these ponds to main reservoirs which also accept gravity drainage from sections of the "new towns."
- o Blending of this urban runoff with runoff from undeveloped watersheds.

The development of this scheme, which has a first stage capacity of 30 imgd and an ultimate safe yield of 60 imgd, included the application of many hydrologic analysis techniques including:

- o Generation of long term rainfall records, using disaggregation techniques, for use in yield studies.
- o Generations of design storm events, both for spillway design events and for the more frequent events where the diversion structures have to operate effectively.
- o Use of a modified version of the STORM model to evaluate the runoff/diversion/pumping operations.
- o Application of the MITCAT model to simulate design event operation of major components of the system.

- o Development of a system simulation model which emulated the operation of the overall collection/storage/treatment system and which was used to:
 - evaluate overall system yield
 - set pump station design rates
 - develop the integrated operation rules for the multiple reservoirs in the system
- o Evaluation of expected water quality in the various components of the system.
- o Application of air-bubble systems to control undersirable stratification in the reservoirs.

The overall scope of the project is to develop a water supply to augment the present supply for municipal uses. Treatment of this water to standards suitable for municipal uses and its discharge at service pressure into the distribution system are included in the project.

A schematic layout of the water supply system is shown in Figure 1. The reservoirs will receive and hold the runoff to equalize the sometimes high and very intermittent runoff rates to meet the steady demand on the system for treated water.

The Seletar Reservoir will be developed by the construction of an earth embankment dam with protecting sluice way across the mouth of the estuary. The reservoir created by the dam will collect runoff which originates on its tributary watershed and runoff from collecting systems located in adjacent catchments. These adjacent catchments are lands which are now being developed into "new towns" and nearby rural areas. A large sand quarry pit will form the Bedok Reservoir. It will primarily receive runoff collected from nearby urban land. Runoff from these catchments will be collected in ponds or lagoons and conveyed to the Reservoir by pumps and pipelines. The water collected in the Seletar Reservoir will be pumped either to Bedok Reservoir or directly to the proposed treatment plant; the pipeline will be about 16km long.

STORMWATER RUNOFF/CAPTURE ANALYSIS

One of the unique features of this scheme is the use of urban runoff as a source of water supply. The drainage system consists of a rather complex group of catchment areas; 23 catchment basins are involved. For twelve of these, flow by gravity into the reservoirs is infeasible and a capture/transfer scheme involving pumping is utilized. The response of this system to rainfall events was evaluated so that overall system yield could be estimated, and the capture/transfer sites could be properly sized.

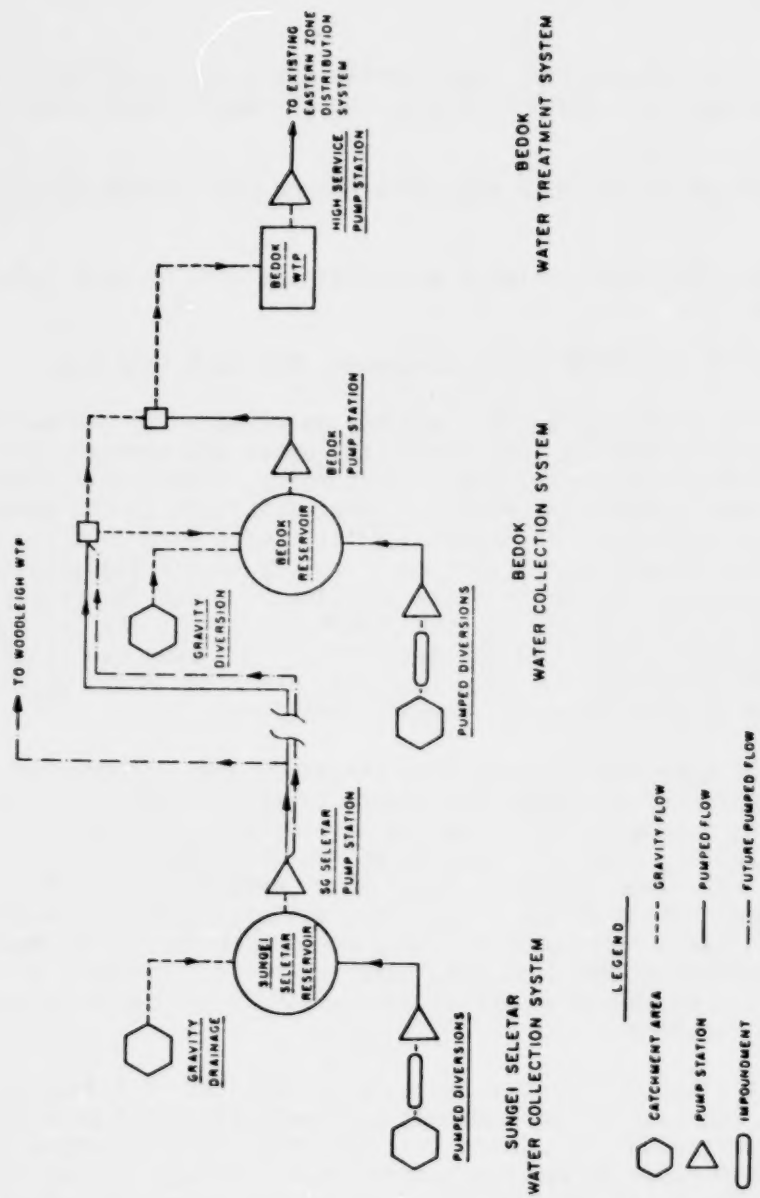


FIGURE 1 : SCHEMATIC OF SELETAR SCHEME

STORMWATER RUNOFF ANALYSIS

The model chosen to simulate the stormwater runoff characteristics of the study area is STORM. In this study, STORM was used to compute surface runoff from precipitation, based on the hydrologic characteristics of the drainage basin.

In order to systematically and consistently apply the STORM Model to each basin, the standard procedure involving the following three steps was employed:

1. Identification of the most appropriate land use categories for the study area.
2. Computation, for each drainage basin, of the area in each land use category, and
3. Estimation of the STORM Model parameters for each land use.

Within the study area, there are a variety of existing or planned land uses. To meet the requirements of this study, the usual six general land use categories were identified as follows: major roads, industrial, institutional, residential, open space, and natural. Some variations in the development levels and thus hydrologic features associated with each land use category exist. However, it was established that the use of six standard land uses adequately represents the hydrologic conditions. Given the rainfall characteristics of the study area and the STORM parameters, the average annual percent runoff was established for each of six land uses and using the standard model parameters for each land use, the STORM Model was systematically employed to simulate basin runoff conditions.

For a number of drainage basins within the study area, stormwater runoff cannot be transferred to system reservoirs by natural means. Such cases require a pumping scheme to lift flows to a reservoir. Rainfall in the area is characterized by relatively high-intensity short-duration storm events. Due to the fairly large amount of impervious surface and the short response time of drainage basins in the study area, this rainfall is transformed into runoff with no large attenuations. This particular condition requires that any runoff transfer scheme involving pumping to also include a stormwater storage facility to retain a portion of the runoff volume for pumping after the storm has subsided.

A schematic of a typical storage and pumping facility is illustrated in Figure 2. In the case shown, stormwater flowing along the main drainage channel is diverted to the storage unit and from there is pumped to the reservoir. When flows exceed the pumping rate, storage is used to contain the excess. When flows are less than the pumping rate, the available pumping rate is used to reduce the storage level. If the storage capacity is exceeded, any excess flow is not diverted to the storage unit.

It is important to note that Figure 2 illustrates one possible storage scheme, which is typically referred to as "off-line" storage. An alternative

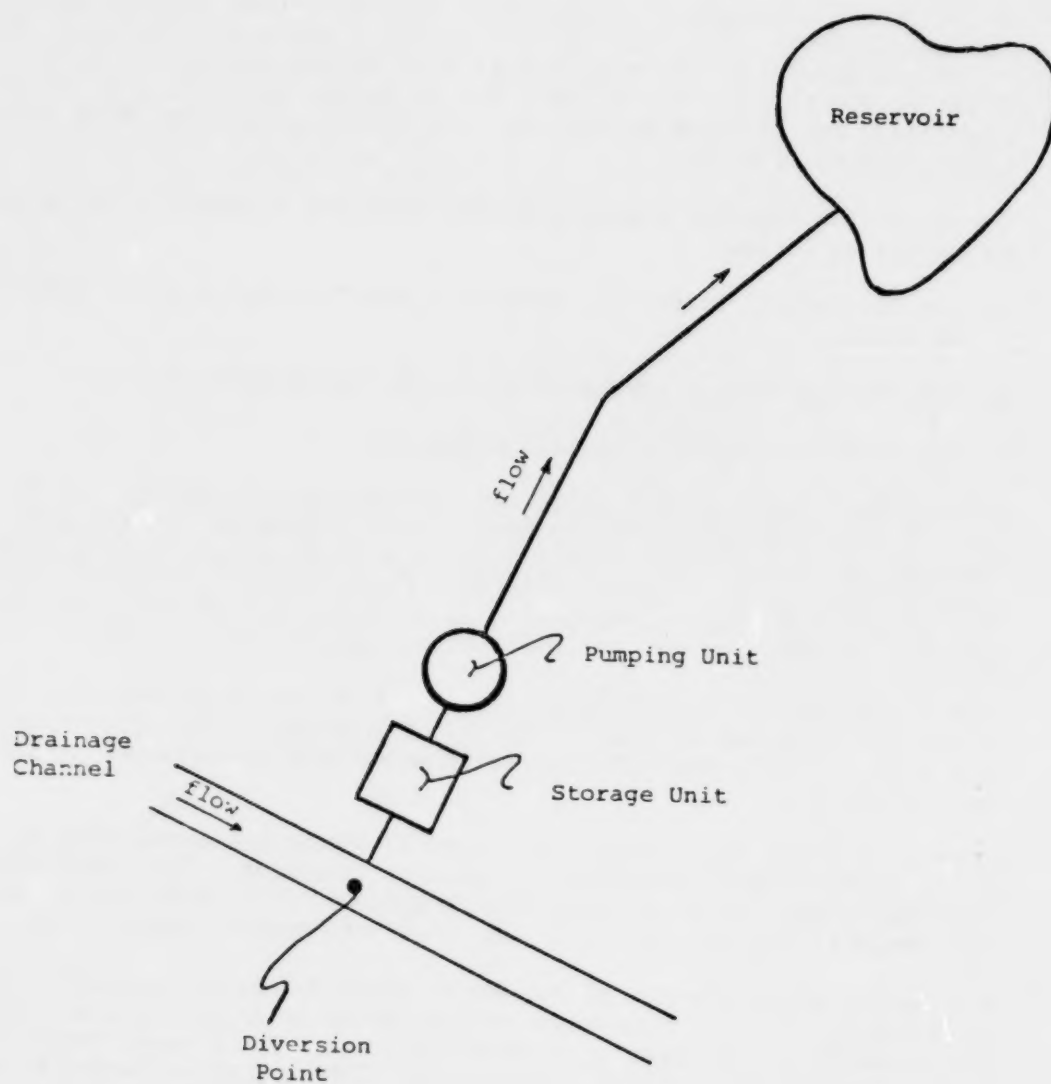


FIGURE 2
SCHEMATIC OF STORMWATER STORAGE AND PUMPING FACILITIES

involves using the main drainage channel as an impoundment for "in-line" storage; pumping occurs directly from the channel. Performance is based only on the storage capacity (i.e., volume) irrespective of its location.

An additional component of a stormwater transfer scheme involves the diversion point itself. The facility must include a means of selectively diverting only a portion of the main channel flow for the reasons as follows: 1) during low-flow periods a bypass may be desired based on water quality conditions, and 2) there may be a maximum hydraulic capacity where excess flow cannot be diverted.

The stormwater transfer scheme described above has a number of basic design parameters as follows:

1. For the channel diversion, there is a low-flow bypass and a high-flow capacity,
2. The storage unit is parameterized by its volume capacity, and
3. The pumping unit has a hydraulic capacity.

Each of these features influences the ultimate capture achieved by the facility. A primary objective of the study was to define the relationships between the design parameters (such as storage volumes) and capture rates. Basic design questions addressed include the following: how much capture is lost to low-flow bypass, what limits do the diversion capacity place on capture, and what are the tradeoffs between pumping and storage?

To evaluate the above methods for each basin in the study area, the DIVERT module for STORM was developed. DIVERT is intended to be a preliminary design tool to assess the hydraulic performance of a large number of process configurations.

Storage-treatment was included in the model due to the anticipated quality of stormwater runoff and potential need for treatment. The DIVERT model was developed primarily to evaluate capture achieved in schemes such as that in this project; however, it has a number of very general capabilities.

A schematic representation of the DIVERT model hydraulic components is shown as Figure 3. System computations are performed on an hourly time step and are typically carried out for a number of years. Basic model input is an hourly time series of runoff generated by STORM, and model output is an hourly time series of system flows. As shown in Figure 3, there are six basic system flows.

Hourly output time series are processed into relevant summaries (i.e., daily, monthly, yearly, and simulation totals) for use in assessing system performance.

The DIVERT model was used extensively in the development of the sizing of the various diversion/pumping alternatives considered. A typical output from the simulation studies is the iso-capture plot shown in Figure 4 where

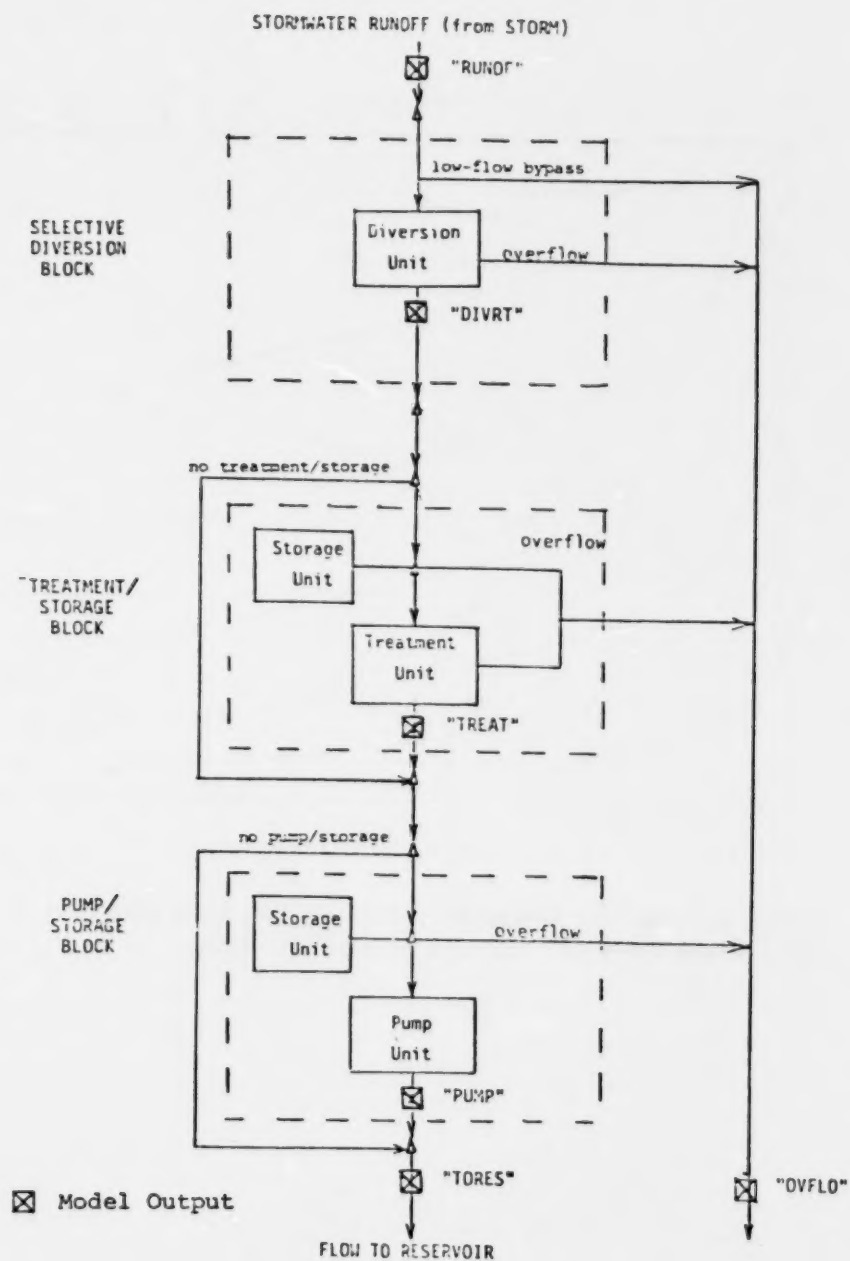


FIGURE 3
 SCHEMATIC DIAGRAM OF DIVERSION MODEL

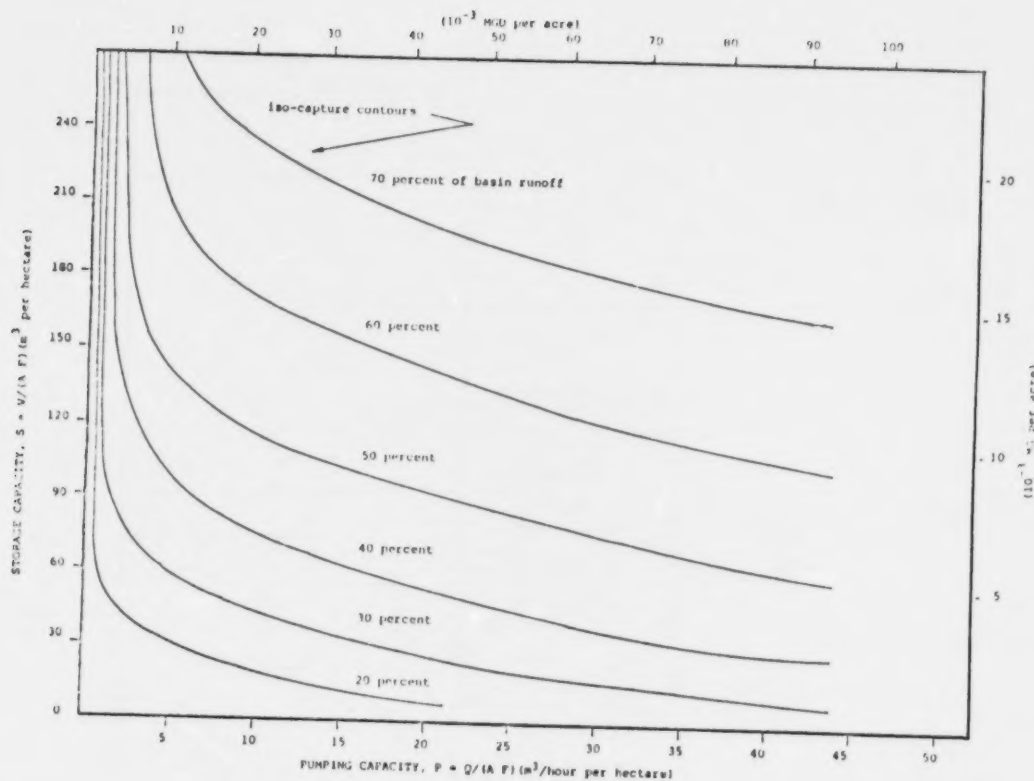


FIGURE 4. CAPTURE NOMOGRAPH, ISO-CAPTURE AS A FUNCTION OF PUMPING AND STORAGE CAPACITY PER UNIT EFFECTIVE AREA

the capture effectiveness of a diversion site is expressed in terms of storage pond volume and pumping rates. Such a plot, combined with the relevant cost data, can then be used to select both the optimum volume/pump capacity at a given site, and also to allocate capture rates between the many sites in use in this scheme. A number of observations can be made from the data shown in Figure 4.

1. Overall, the spacing between iso-capture contours steadily increases, indicating an increasing marginal capacity required for additional capture.
2. Beyond 50 to 60 percent capture, the marginal capacity requirements become relatively large.
3. Along the vertical axis (i.e., constant storage volume), there is an abrupt increase in the marginal pump requirements, and beyond about 3 to 5 m³/hour per hectare very large additional pump capacity is required for further capture.

These figures are specific to hydrologic conditions for this scheme; similar figures can be readily estimated for other sites.

SELECTIVE DIVERSION

Selective diversion potentially consists of two components - a low-flow bypass and high-flow diversion capacity. It is assumed that the bypass must be satisfied before diversion begins and further that a limited hydraulic capacity at the diversion structure exists such that excessive flows cannot be routed to the pump/storage unit. Given this condition, the STORM/DIVERT model was used to determine the relationship between the quantity of runoff diverted and the diversion structure parameters. The following general observations can be made about the results obtained during this study:

1. Overall, there is a steady decrease in the sensitivity of capture to increases in bypass and diversion capacity.
2. For relatively low bypasses there is a very rapid loss in capture effectiveness.
3. Beyond a certain maximum diversion capacity very little additional capture is achieved.

Bypass will be used primarily to exclude low-flows which may contain unacceptable water quality constituents. From a capture point of view, results indicate that a relatively large amount of runoff can potentially be lost via bypass flows, and thus only the minimum required to satisfy water quality requirements should be excluded. These are flows which are easily captured from a hydraulics point of view and should not be bypassed unless clearly necessary from a water quality perspective.

WATER QUALITY ASPECTS

Numerous water quality issues are important to the implementation of the proposed scheme. These issues are primarily stormwater runoff quality loadings and salinity and TDS levels in the reservoirs. Stormwater runoff quality is of importance since the water supply system consists primarily of this urban runoff; these flows are the main source of nutrients to the reservoirs.

In general, storm runoff is characterized by relatively high pollutant loadings; these pollutants exist and are washed off from both developed and undeveloped land surfaces. In this particular project, increased emphasis was placed on runoff quality estimates because of the unusual nature of the scheme as a whole. A relatively straightforward approach was taken with a somewhat modified version of the STORM model being used in the simulation of runoff quality loads. Available local historic hourly rainfall records were used to drive the STORM simulations, with a three year period of record being found sufficient to provide reasonable estimates of long term averages; generally a ten year hourly record was available for use. An important set of parameters in modeling runoff quality is the pollutant accumulation and washoff rates for the specific area being studied. Ideally, sampling data from the area would be used to calibrate these parameters of the simulation model. Although specific data of this nature was obviously not available from the yet-to-be-developed "new towns," some data was available for nearby similar developments. This data, coupled with values available in published literature was sufficient to enable the team to estimate the long term average loadings to Total Suspended Solids, Nitrates, Phosphates and Biological Oxygen Demand resulting from the stormwater runoff. Sensitivity studies were undertaken to determine the impact of parameter uncertainty on the loading estimates; the design of the overall system is sufficiently robust to adequately handle expected pollutant variations.

The modifications of the STORM model were primarily to enable the model to:

1. Simulate the transfer of pollutants through the diversion components including the actual diverters, settling of solids in the small ponds, and the pump operations.
2. Evaluate the operation of swirl concentrators which were considered for use at the diversion sites. These were ultimately not recommended for use.

The runoff from the catchment areas is captured and stored in the reservoirs prior to treatment. The benthos deposits in the reservoirs are also a major source of contaminant loadings in the system; in previous similar schemes they have been a significant operational nuisance. It is expected that phosphate, chloride and sulphates will be the contaminants of most importance in this scheme.

SUMMARY

This application required integrated use of many hydrologic modeling techniques in the development of a highly complex system. Their use in this project permitted significant increases in the design "safe yield" of the system and enabled the design team to evaluate the effectiveness of the many quantity and quality control options which have to be considered during design development. Problems do exist in the use of urban runoff for supply systems; however, in situations where well-designed separate storm drain systems are available, the runoff can be effectively used as a supply source.

JEN AI RIVER - KAOHSIUNG, REPUBLIC OF CHINA

BACKGROUND

Kaohsiung is a city of about one million people located on the southwest of the Island of Taiwan.

There are now essentially no sanitary sewers in Kaohsiung. All sanitary wastewater, together with stormwater, flows via combined sewers and open channels to the Jen Ai River, a narrow, poorly-flushed tidal estuary. The resulting very heavy loadings of BOD have rendered the Jen Ai anoxic; the river is both odorous and an eyesore due to floatables.

The city government is presently undertaking a project to radically improve the water quality in the Jen Ai. This project has several major components as follows:

1. Build a trunk sewer of sufficient capacity to intercept all the dry weather flow (DWF) and some of the wet weather flow at each of 8 principal tributary channels or sewers.
2. Remove the majority of the anoxic sediments from the bottom of the Jen Ai by dredging.
3. Implement other means to maintain an improved water quality in the river.

At this time, a trunk sewer has been constructed to intercept combined storm and sanitary sewage from 8 catchment areas, 6 on the east bank of the Jen Ai River, and 2 on the west bank (see Figure 5). The area drained by the 8 basins is about 5400 ha. The total dry weather flow from the 8 areas is 3 cms, and the total dry weather BOD load is 56000 kg/day.

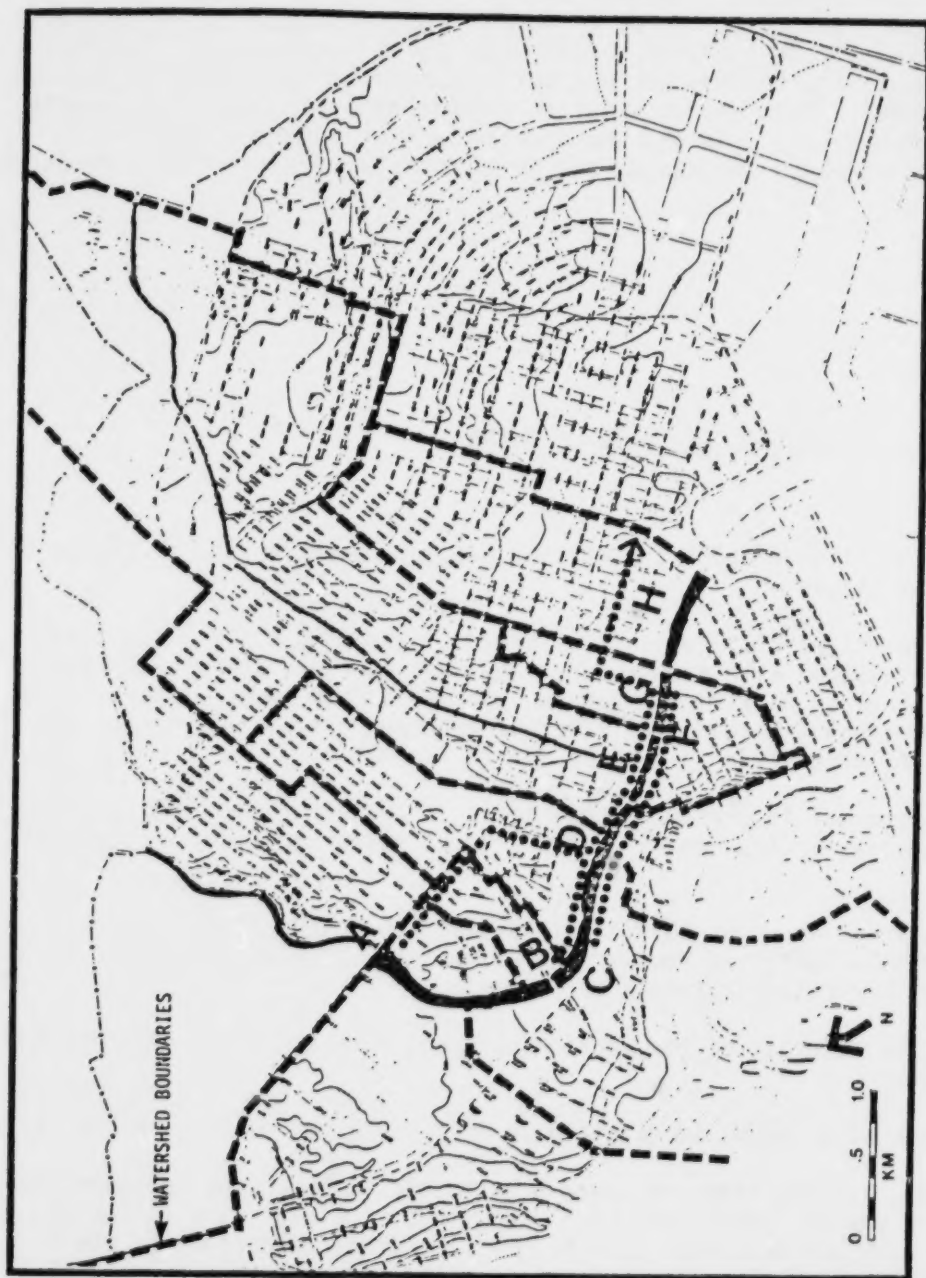


FIGURE 5. MAP OF JEN AI RIVER AND INTERCEPTED TRIBUTARIES

The total design capacity of the trunk sewer and the pump station at its downstream end is 6.5 cms, or about twice the combined total dry weather flow of the 8 areas. Thus all dry weather flow and some storm flow will be intercepted, but stormwater will still overflow to the river when flows exceed the trunk sewer capacity.

The analytic work described here was undertaken to evaluate the effectiveness of the proposed scheme in terms of improvements in D.O. in the Jan Ai River, and to assist in the sizing of the diversion structure. On the larger tributaries, the provision of capacity to store significant quantities of stormwater, until the water can be drained to the sewer following the storm, greatly reduces the frequency and magnitude of storm overflows, and the mass of BOD discharged in overflows.

On the smaller tributaries with little or no storage capacity, where much of the stormwater will have to overflow to the river, the use of swirl concentrators to remove TSS and BOD from overflow water before its discharge to the river was evaluated.

As will be noted, the plan is to divert a significant fraction of the total BOD loads from the overall watershed through use of quite a small interceptor, with essentially the whole drainage system diverted through it during dry weather conditions. The storm discharges from the basin are about 63 cms for a "typical" monthly storm (6mm/hour) and 540 cms for the 5-year design event; these figures should be contrasted with the 6.5 cms peak capacity of the interceptor.

APPLICATION OF STORM

The STORM model was used extensively in the following aspects of the study:

- (1) To determine runoff rates from the various watersheds.
- (2) To determine mass loadings of BOD and TSS from the area.
- (3) To assist in sizing the various control facilities required to divert flows.
- (4) To help evaluate long term management strategies for the basin.

Hourly rainfall data was available for 12 years; typical simulations used 6 years of records which included a typical range of wet and dry years. The climate of Kaohsiung is characterized by an extremely dry winter from November through March followed by a very wet summer season.

Solids buildup in the side drains and street-side ditches tends to be significant during the winter months; during this period the flows in the system are largely sewage and grey water. Little or no flushing of the estuary occurs during this period.

The months of April and May are characterized by moderately intense rain-falls; these first storms after the dry season flush much of the deposited solids from the network and dump them in the river. The assimilative capacity of the river during this period is overwhelmed by the total magnitude of the BOD loads.

A modification was made to the STORM model to enable us to simulate the long term deposition of solids in the "sanitary" system since the system is not self cleansing as in typical U.S. systems. These depositional factors, F1 and F2, are the mass fraction of the TSS and BOD in sanitary sewage which in dry weather settles out of suspension and is deposited in the drains and watercourses. These deposits are then resuspended and flushed to the interceptor/river during storms. It was not possible to perform field tests to determine the actual deposit fraction for TSS and BOD during the course of the project. The values used for most STORM runs were 0.7 for TSS and 0.3 for BOD. These values are probably conservatively high since channels at low-flow sites cannot be assumed to be as efficient as primary clarifiers would be. However, since our concern was with the fraction resuspended during the storm events, it was decided to adopt the above figures.

The overall system was evaluated by simulating each individual basin and its interceptor junction characteristics; the allocation of interceptor capacity against basins was as further indicated below.

The results of the simulation runs were largely as anticipated. Figure 6 shows the impact of allocated interceptor capacity on the BOD overflowing to the river from a typical basin; it will be noted that the allocation of flow capacity above the DWF results in only minor reductions in overflows. It proved possible to use disused canals as storage basins in several instances; the impact of such storage on BOD overflows can be significant and as shown in Figure 7 can reduce the fraction of BOD overflowing to the river from about 30% to 5% of total BOD loads from the tributary basin.

The impact of using swirl concentrators for grit and solids removal at several of the smaller basins is shown in Figure 8. Although effective on a single site basis, they did not prove cost effective in terms of the overall Jen Ai control strategy.

Schematic layouts of a typical interceptor structure are shown in Figure 9-11.

Overall, it is estimated that the proposed scheme will reduce BOD loads to the Jen Ai River by about 90% from existing conditions. Overflows will still occur about 20-40 times per year; many of the overflows are quite small and the BOD loads minimal.

The computed reduction in BOD loading to the river is just about adequate to enable the estuary to achieve a D.O. level of 2 ppm under average month conditions. Heavy loadings in the spring months will be sufficient to de-oxygenate the river during these periods. As a result, it is proposed that a series of air-bubblers be installed to ensure that the estuary remains fully mixed and surface oxygenation maximized. Simulations indicate that

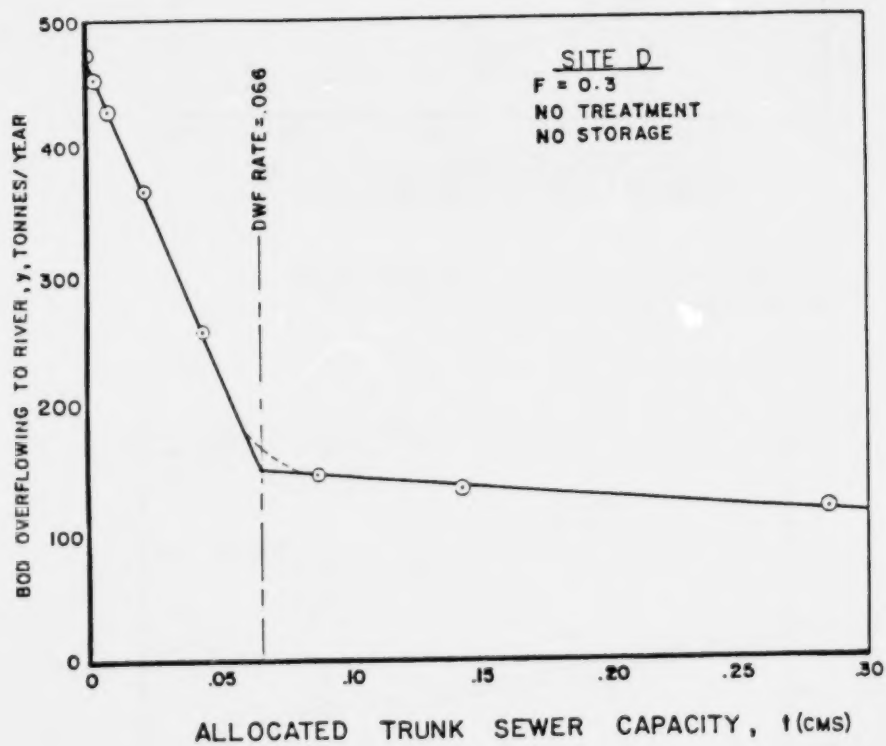


FIGURE 6. INFLUENCE OF TRUNK CAPACITY, t ,
ON BOD OVERFLOW RATE, y

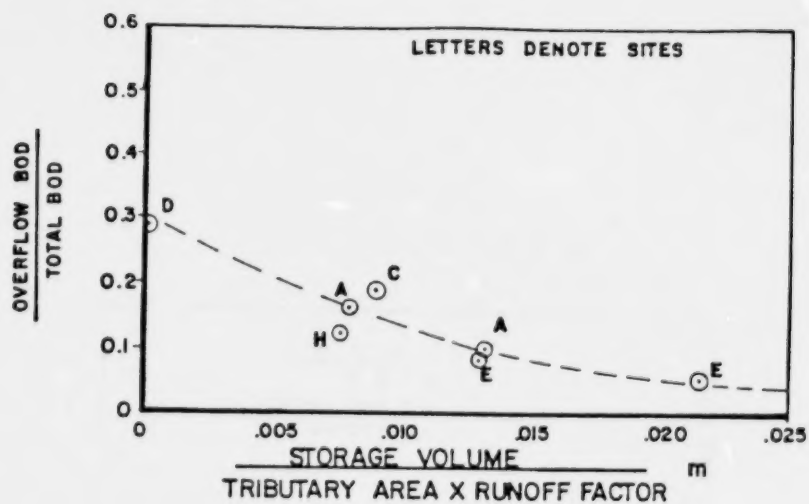


FIGURE 7. INFLUENCE OF STORAGE VOLUME ON BOD OVERFLOW RATIO

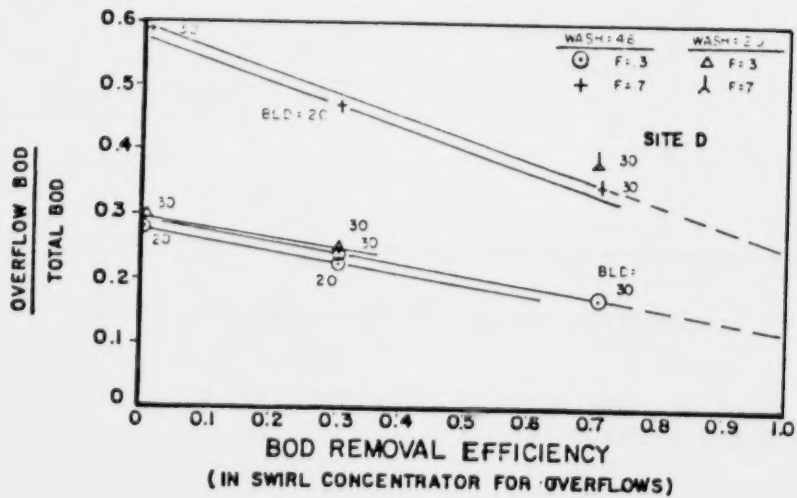


FIGURE 8. INFLUENCE OF BOD REMOVAL EFFICIENCY ON BOD OVERFLOW RATIO

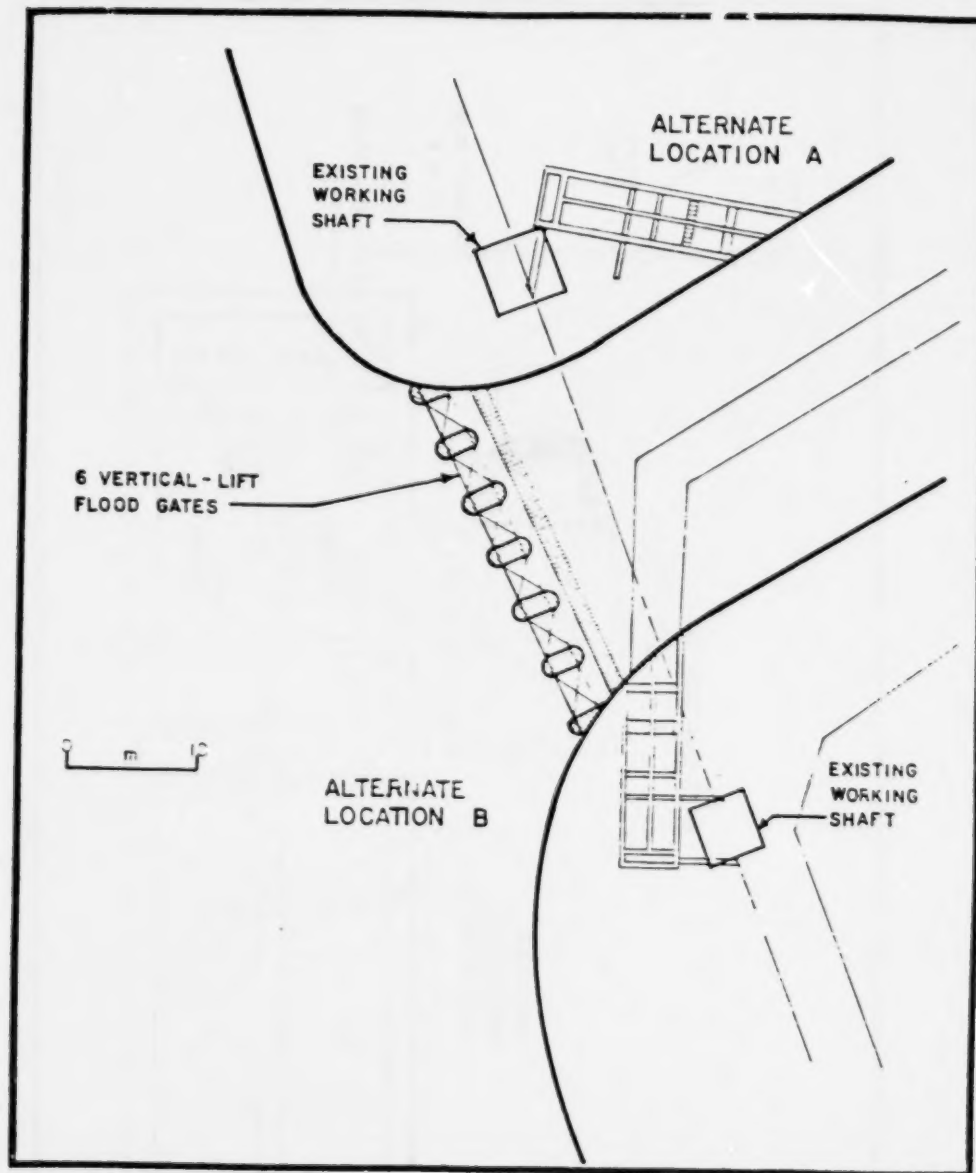


FIGURE 9. SITE PLAN FOR INTERCEPTOR STRUCTURE AT "E". TWO ALTERNATE LOCATIONS ARE SHOWN.

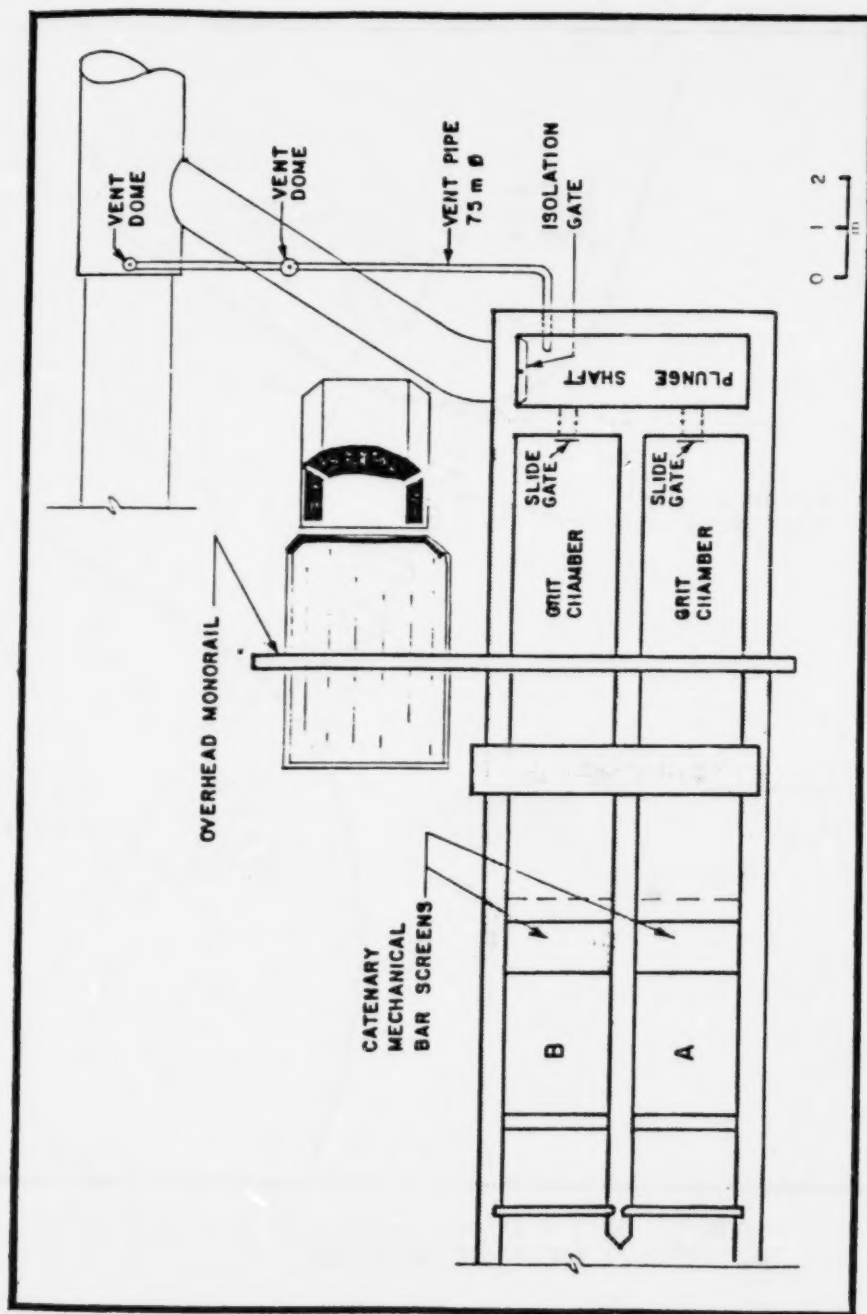


FIGURE 10. PLAN FOR INTERCEPTOR STRUCTURE FOR "E"

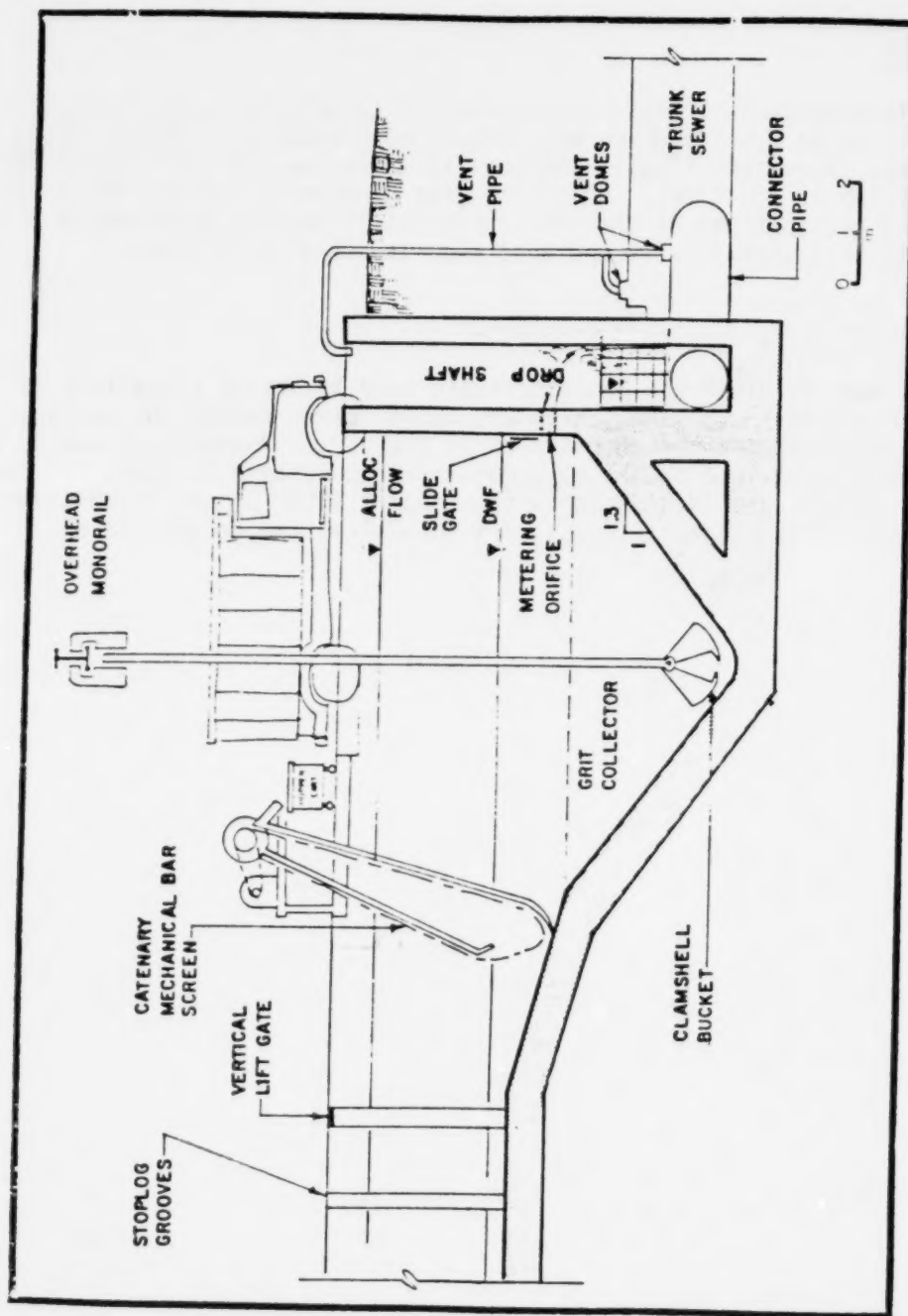


FIGURE 11. PROFILE FOR INTERCEPTOR STRUCTURE AT "E"

this will maintain adequate D.O. levels in the river.

SUMMARY

In this situation, it has proved possible to quickly restore adequate water quality to an estuary which was grossly overloaded with uncontrolled sanitary and stormwater flows. The cost is quite low and well within the means of developing countries. Of course, the problem is not yet fully solved - those areas upstream of the interceptor still contain open sewers which will have to be systematically upgraded over the next 10-20 years.

CONCLUSIONS

Models such as STORM can be effectively used in design situations in developing countries. In many instances, the problems facing the designer are much tougher than those encountered in the U.S. - fundamental public health improvements must be sought with moderate expenditure of funds for construction. In the cases discussed in this paper, STORM proved to be a very effective tool in assisting the designers to achieve the design goals.

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

AN ATTEMPT TO IMPLEMENT SWMM IN TUNISIA

by Janusz Niemczynowicz

ABSTRACT

In 1980, the University of Lund started a research project in co.operation with the University of Tunis. The aim of the project was to implement the SWMM in Tunis and to teach local research personel how to handle the model. During the winter season, there are flooding problems in the cities of Tunisia due to high-intensity convective rain storms and underdesigned sewage systems. The greatest efforts in the city of Tunis until now have been made to solve the immediate problems as street flooding by constructing a huge stormwater conduits. This will probably increase already severe pollution problems in the receiving waters. It is obvious that some kind of system thinking has to be applyed in this situation. Implementation of SWMM-model was thought to be one of the masures possible to take in order to assure meaningfull water management in th cities. In order to obtain the input data for calibration of the model, a catchment of 20 sq km was instrumented. Differences between Swedish and Tunisian urban areas, together with the climat in Tunisia, couse signifficant differances in the input parameters in the model. For example, roughnes in water courses can not be treated as a constant in Tunisia. During the dry period great amount of garbage is accumulated in the water corses, during rainy season the garbage and lush vegetation is succesively removed and the roughnes decreases gradually. The paper describes how the difficulties during field work, differances in working routines, problems with understanding the "modeling philosophy", and problems with model calibration influence the possibility of model implementation in Tunisia.

INSTRODUCTION

The average, yearly precipitation in Tunis is about 450 mm. The major part of the yearly rainfall occurs during October, November and December, usually as short, intensive convective rainstorms. These rainstorms often take the form of thunderstorms. The rest of the year, there is very little precipitation. Tunis is surrounded by high hills which slope steeply towards the central part of the town causing severe problems with storm water management.

The storm water system presently existing in Tunis is not sufficiently designed to handle storm water runoff resulting in street flooding several times a year. During the past three years, great efforts have been made to reduce flooding by constructing a huge storm water conduit which leads water from the central part of the town to the nearby situated Lac de Tunis lagoon (Coyne 1974). It is unlikely that even these efforts will significantly improve the situation due to rapidly increasing urbanisation of the suburban areas. Other problems will occur, i.e. pollution of the lagoon by storm water discharge.

Most of the cities in Tunisia has the same problems. It is obvious that some kind of system thinking has to be applied in this situation in order to assure reasonable way to solve the design problems associated with storm water systems. Implementation of SWMM-model was thought to be one of the measures possible to take in order to assure meaningful water management in the cities avoiding all the mistakes previously done in European countries in this matter.

In 1980, a research project was started in co-operation with the University of Tunis. The aim was to implement the Storm Water Management Model and to teach local Tunisian research personnel how to handle this model.

In order to calibrate the model, a large amount of rainfall and runoff data had to be collected in an environment rather different from the Swedish one. We decided to use the 20 km² Guereb-Roriche catchment in the northern part of Tunis.

CATCHMENT AND INSTRUMENTATION

The Guereb-Roriche catchment consists of two areas: the highly urbanised section in the south and the rural area in the north with a few single-family dwellings. (See Figure 1).

The urbanised area of the catchment is densely populated and housing consists of multi- and single-family buildings and some industries. The character of

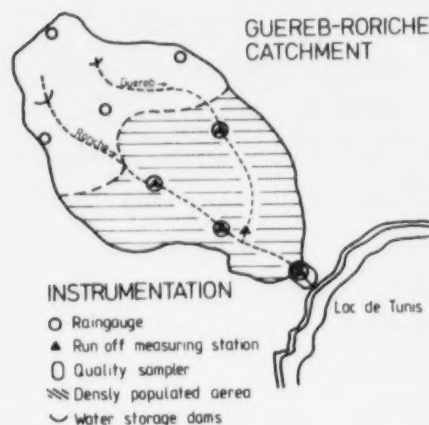


Figure 1. Instrumentation of the Guereb-Roriche catchment.

the housing is quite different from European housing, which has significant hydrological consequences. For example, almost all houses have flat roofs with elevated edges, which causes roof ponding. Some of the minor streets are not paved. Roof drainage is rarely connected to the storm water system. The percentage of impermeable surfaces is very high - up to 60% in some areas.

The southern part of the catchment is very flat and varies less than 50 meters in altitude.

The northern part, on the other hand, is hilly with altitude differences up to 250 meters. Agriculture dominates in this rural area but the uppermost region is uninhabited due to severe erosion of the steep hillsides.

The catchment is drained by two periodically appearing rivers, the Guereb and the Roriche, which join in the lower part of the catchment and then flow into the Lac de Tunis lagoon. In the upper part of the catchment, there is no flow in either river during the dry period. However, there is some baseflow occurring in the lower part due to the release of waste water from the slum

areas. During the wet season, discharge from both rivers can be as high as 15 m³/s.

The soil type within the catchment is laterite clay. Because of cracks occurring in this clay during the dry season, it becomes very permeable. After the first few rainfalls, permeability of the clay reduces drastically (Infiltration tests made during the dry season have shown that the infiltration capacity exceeds 50 inches per hour during the next three hours). It can be expected that infiltration tests made during the wet season will give quite different results.

Urbanisation of the rural northern regions is proceeding rapidly and it is expected that the entire catchment will be urbanised within a few years. The development plans for the northern part include single-family homes, high multi-family buildings and schools.

In order to take into account the nonhomogeneous character of the catchment, and achieve rainfall areal characteristic, the runoff and rainfall measuring stations were installed in 9 field stations (Figure 1). (Niemczynowicz et al 1981).

As the distance between the stations is rather long, and there are no reliable telephone lines available, it was decided that all stations should be equipped with separate clocks, data-loggers and automatic start-/stop-units.

The rainfall gauges are of the standard tipping-bucket type manufactured by Rimco, Australia, with a bucket capacity of 0.2 mm. The funnel diameter is 203 mm and the total accuracy is $\pm 1\%$ up to 380 mm/h.

The runoff gauge is an echosounder manufactured by Endress and Hauser, West Germany. It permits a non-contact continuous measurement of the water level over the weir.

In addition, there is also a mechanical water level recorder ensuring that no data are lost during current-failure periods.

The datalogger, manufactured by A D Data Systems Inc., USA, Type ML-10, has an internal quartz-controlled clock and a capability of monitoring up to 10 analogue signals and 32 bits of digital data. The data are recorded in a computer-compatible format on a Philips-type data cassette. Data can be recorded at pre-selected intervals which are controlled by the digital clock, by a front-panel manual switch or by external input.

The recording interval is 5 minutes.

The automatic start/stop unit offers two possibilities of starting a recording sequence.

If the water rises to a certain pre-selected level, the recording starts; if the raingauge counter reads two or more, the recording is also started. An additional timer keeps the recording for 2 hours after the last raingauge tipping to make sure that the whole hydrograph is recorded.

The stations operate on a 220 V 50 Hz supply, but as we have had som bad experiences with the power supply, especially during rainfalls, we have changed the stations to battery power.

Figure 2 and 3 show two of the runoff measuring stations.



Figure 2 Runoff station



Figure 3 Runoff station.

RESULTS

During 1981 and 1982, great efforts were made to collect and analyse available documents and maps in order to gather all data necessary for modeling. We often discovered that important data were missing or could not be delivered by the officials in charge. There were also many maps of conduits that did not correspond to the actual situation in the field. As a result, a lot of field work had to be done for verification of necessary data.

The catchment was divided into forty-six subcatchments ranging in size from 0.06km^2 to 1.44 km^2 (see Figure 4). The storm water system consists of seventy-seven sewer pipes and two storages in the upstream region of the catchment (see Figure 5).

After two years of data collection only about 8 rainfalls-runoff occurrences were registered on more than one station. Only service in the field which had to be done to assure registration was changing the cassettes, paper charts and the batteries in adequate periods of time. Unfortunately, most of the high intensity rainfalls happen to occur while batteries were empty or cassettes were full. During the third and final season of registrations

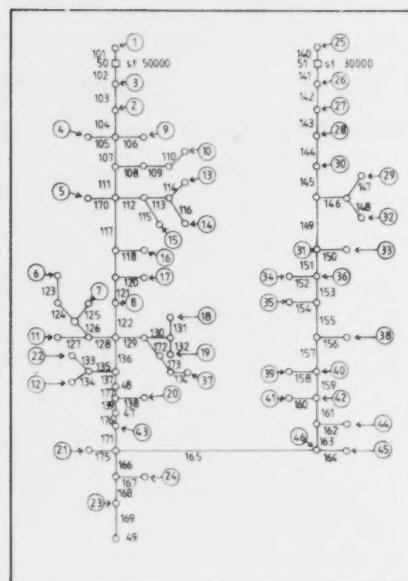
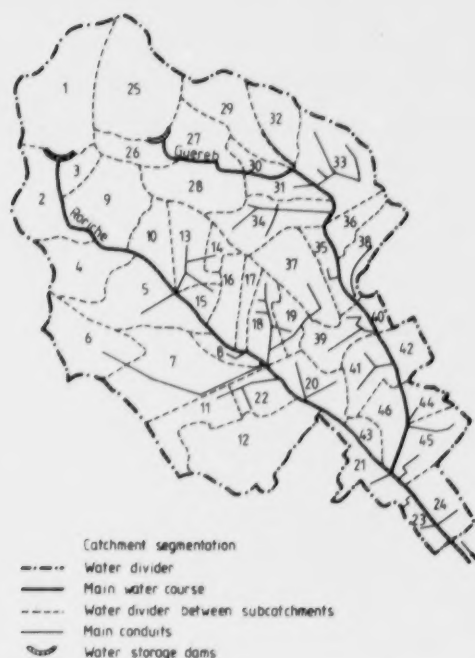


Figure 4 Guereb-Roriche catchment in Figure 5. Guereb-Roriche catchment.
Tunis Diagram of storm water system.

the schedule for permanent superintending by swedish personnel was established. This resulted in that sufficient amount of field data to make a reasonable calibration of the model was collected.

MODEL CALLIBRATION

In co-operation with two students from the University of Tunis, an attempt was made to calibrate the model. We discovered that our initial estimates of the parameters were rather inaccurate.

Fig 6 shows an example of the callibration runs. The first run was completely

different from observed hydrograph. We soon discovered that procentage of impermeable surfaces taken directly from the maps does not represent the real situation. Only about 40% of all impermeable surfaces is connected to storm water system. The main problem however is that this figure changes with time during the rainy season. The reason is to find in erosion of soil between paved surfaces, which gradually open small channels connecting new areas to the system. One of two parameters depression storage or procentage of impermemeable surfaces must to be treated as a function of time.

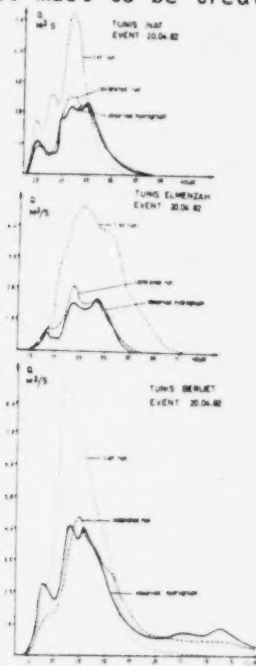


Figure 6 Examples of observed and SWMM-simulated hydrographs

The width of the overland flow (w -parameter), which influences the shape of the hydrograph, calculated accordingly to the generally accepted rules, appeared to be far too high. This can be explained by the fact that the surfaces are more undulated than we are used to, which makes the length of the overland flow longer. Because of the high values of the infiltration parameters, used in the first simulations, no runoff was simulated from permeable surfaces. This is probably true for some of the first rainfalls in the beginning of the rain season. During the rain season, the infiltration capacity changes drastically and the infiltration parameters should be changed

accordingly. During the first rainfalls, SWMM overestimated the peak flow and runoff volume, but during the following weeks, the simulations were much better (see Figure 7). This can probably be explained by the variations of the roughness in the main water channel. The river Guereb and Roriche, which drain the catchment, have rather flat sloping sides in the lower part of the catchment. During the dry season, very lush vegetation covers the slopes of the channel and most of the vegetation from the side slopes changing the roughness of the channel drastically.

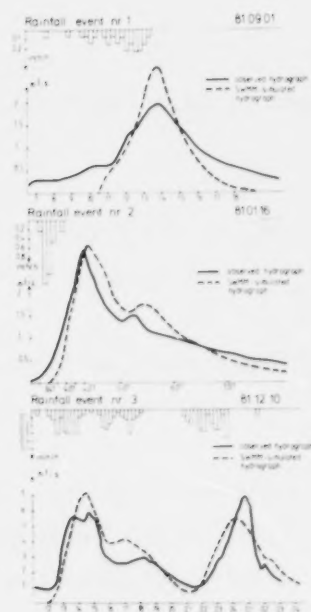


Figure 7 Examples of observed and SWMM-simulated hydrographs

In spite of those difficulties and differences in the magnitude of model parameters it is possible to reproduce the observed hydrograph quite well. The areal distribution of rainfall has to be taken into account because differences in rainfall intensity are very significant.

The one of the goals of the projekt was to implement SWMM on the French computer in the university of Tunis. The Guereb-Roriche catchment

was ment to be only a experimental field for demonstration of model application, and for training Tunisien personnel in geting indata, running the model and making design simulations. With other words, the project was thought to be demonstration project to teach "modelling philosophy". The main problems which we faced in this context was the lack of hydrologists simultaneously able to run the computer. On the other side there are no computer man who understand hydrology.

Until now all the computer runs were done in Sweden.

REFERENCES

- Coyne et Bellier, 1974, Etude de l'assainissement de Grand Tunis. Etude de facilité du plan directeur, ICN, Pays-Bas.
- Dahlblom, P; Niemczynowicz, J; Hogland, W, 1982, "Significance of Water Planning in Tunisia". Dept of Water Res. Eng., Lund University, 1982.
- Dendrou, S A ; Delleur, J W, 1978, Planning Storm Drainage Systems for Urban Growth, Journal of the Water Resources Planning and Management Division, Nov.
- Hogland, W; Niemczynowicz, J, 1980, Kvantitativ och kvalitativ vattenomsättningsbudget för Lunds centralort. Kompletterande mätningar och metoder. Institutionen för teknisk vattenresurslära, LTH/LU, Nr 3038, Lund.
- Niemczynowicz, J; Grahn, L, 1981, Hydrological Instrumentation. Fifteenth Anniversary Report, Department of Water Resources Engineering, University of Lund, Report No 3053.

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

THE SEDIMENT-WATER INTERFACE IN MODELING PESTICIDES IN SEDIMENTATION PONDS

by

Peter R. Jaffe and Raymond A. Ferrara

Department of Civil Engineering
Princeton University
Princeton, NJ 08544

Introduction

Many pesticides adsorb onto soil particles upon application to fields. Stormwater runoff then results in the transport of these soil particles with adsorbed pesticides to the receiving waters. As a result, peak pesticide concentrations, sometimes reaching acute levels, can occur in receiving waters during storm events that follow pesticide application. Since particulate material can be removed via sedimentation, the objective of this work is to study what effect a sedimentation pond may have on the pesticide load to receiving waters.

The model presented in this study considers that the pesticide fraction adsorbed onto the sediments will be removed during the sedimentation process, and incorporated into the bottom sediments. When after several storm events the dissolved pesticide concentration in the overlying water column decreases, the bottom sediments will release some of the pesticide contained in them. The sedimentation pond will then behave like an equalization basin, capable of buffering peak loads. This buffer capacity is enhanced by the sedimentation process, and the capacity of the bottom sediments to first retain and then release again a portion of the applied load. Furthermore, if decay occurs, a pond could be designed to provide sufficient detention time to decrease the net total load to the receiving water.

Model Development

The water column of the pond is represented as a completely mixed system. Pesticides are added to the pond during storm runoff in dissolved and adsorbed phases. These phases are at equilibrium as defined by their partition coefficient (i.e. a linear Freundlich adsorption isotherm). The dissolved phase can undergo any of a series of first order transformation processes, such as chemical or biological degradation or evaporation, all of which are additive. Other sources and/or sinks of the dissolved phase are the equilibrium interaction with the phase adsorbed onto sediments, and diffusion into or out of the bottom sediments. The adsorbed phase can also undergo any of a series of first order transformation processes in addition to settling from the water column to the bottom sediments. Consequently, to model this system, equations are required to describe the changes in

volume, suspended sediments, adsorbed pesticide concentration, dissolved pesticide concentration, and pesticide concentration in the bottom sediments.

a) change in volume

$$\frac{dV}{dt} = Q_i - Q_o + A(P - E) \quad (1)$$

$$Q_o = K L \left\{ \frac{V - VM}{A} \right\}^{2/3} \quad (2)$$

if $V \leq VM$, then $Q_o = 0$

$$\frac{dVM}{dt} = - R \quad (3)$$

where:

A = surface area,
 E = evaporation,
 K = constant,
 L = length of the weir,
 P = precipitation,
 Q_i = inflow,
 Q_o = outflow,
 R = change of volume due to the deposition of sediments,
 V = volume, and
 VM = holding capacity of the pond.

b) suspended sediment

Many chemical and physical properties of individual sediment particles are dependent on their size distribution (e.g. clay has a higher adsorptive capacity and a lower settling velocity than sand). Consequently the size distribution of sediments contained in storm water runoff is important. Furthermore irregularities in shape require the specification of a settling velocity distribution for particles which are nominally of the same size (Ferrara and Salvage, 1982). Then:

$$\frac{d}{dt} (C_{sjk} V) = Q_i C_{sijk} - Q_o C_{sjk} - A v_k C_{sjk} \quad (4)$$

where:

C_{sjk} = suspended sediment concentration of j^{th} size fraction with settling velocity v_k ,
 C_{sijk} = C_{sjk} in the inflow, and
 v_k = settling velocity.

then the term R in Equation (1) is

$$R = \frac{A}{\rho_s(1-\phi)} \sum_j \sum_k v_k C_{sjk} \quad (5)$$

where: ρ_s = density of an average sediment particle, and
 ϕ = connected porosity.

The sediment-size distribution is divided into n size fractions, $j = 1 \rightarrow n$. Each size fraction has a velocity distribution that is divided into m velocities, $k = 1 \rightarrow m$, but m does not have to be the same for each j . For non-settling solids (colloids), the velocity is zero.

c) adsorbed pesticide concentration

The change of concentration with respect to time of adsorbed pesticides in a given size-velocity fraction of the sediments in the water column is:

$$\frac{d}{dt} (C_{ajk} V) = Q_i C_{aijk} - Q_o C_{ajk} - A v_k C_{ajk} - V k_j'' C_{ajk} - D_{jk} \quad (6)$$

The overall adsorbed concentration is then

$$\begin{aligned} \frac{d}{dt} (V \sum_j \sum_k C_{ajk}) &= Q_i \sum_j \sum_k C_{aijk} - Q_o \sum_j \sum_k C_{ajk} - A \sum_j \sum_k v_k C_{ajk} \\ &\quad - V \sum_j \sum_k k_j'' C_{ajk} - \sum_j \sum_k D_{jk} \end{aligned} \quad (7)$$

where: C_{ajk} = adsorbed concentration on j^{th} sediment fraction with settling velocity v_k ,
 C_{aijk} = C_{ajk} in the inflow,
 k_j'' = sum of first order reaction rates of the adsorbed pesticide, and
 D_{jk} = desorption term.

d) dissolved concentration

The equation describing the dissolved concentration in the water column is:

$$\frac{d(V C)}{dt} = C_i Q_i - C Q_o - k' V C - A J + \sum_j \sum_k D_{jk} \quad (8)$$

and

$$J = - \phi (D_B + D_S + D_I) \frac{\partial C}{\partial x} \bigg|_{x=0} + \phi v_w \bigg|_{x=0} \quad (9)$$

where:

- C = dissolved pesticide concentration,
 k' = sum of all first order reaction rates,
 J = losses into the bottom sediments across the sediment-water interface (advective plus diffusive flux),
 D_B = bioturbation coefficient,
 D_S = molecular diffusion coefficient,
 D_I = irrigation coefficient,
 v_w = velocity of water burial below the sediment water interface, and
 D_{jk} = desorption term.

The equilibrium between the dissolved and adsorbed phases is

$$C_{a_{jk}} = K_j C C_{s_{jk}} \quad (10)$$

where K_j = partition coefficient.

Substituting equation (10) into equation (7) to eliminate the adsorbed concentration and then combining this equation with that for the dissolved concentration, equation (8) eliminates the desorption terms and yields the final equation for the dissolved pesticide:

$$\frac{dC}{dt} = \frac{Q_i C_i \sum_j (K_j C_{s_{ij}}) - C[Q_0 + V k'_j] \sum_j (K_j C_{s_j}) - AC \sum_j v_j K_j C_{s_j} + C_i Q_i}{V[1 + \sum_j K_j C_{s_j}]}$$

$$+ \frac{C Q_0 - k' V C - A J - \left\{ C[1 + \sum_j (K_j C_{s_j})] \frac{dV}{dt} + V C \sum_j (K_j \frac{dC_{s_j}}{dt}) \right\}}{V[1 + \sum_j K_j C_{s_j}]} \quad (11)$$

In summary, the equations that have to be solved for the water column are:

- 1 equation for the volume,
- 1 equation for each size-velocity fraction of the sediments, and
- 1 equation for the dissolved pesticide.

The total adsorbed pesticide can always be estimated as:

$$C_a = C \sum_j \sum_k K_j C_{s_{jk}} \quad (12)$$

and the total pesticide concentration (adsorbed and dissolved) as:

$$C_T = C(1 + \sum_j \sum_k K_j C_{s_{jk}}) \quad (13)$$

Solution of this system of equations requires knowledge of the dissolved concentration gradient in the bottom sediments (equation (9)). Therefore solution of the concentrations in the bottom sediments is also necessary.

e) bottom sediments

The development of the following equations is summarized from the work of Berner (1980).

The mass balance equation in the sediments is:

$$\frac{\partial \hat{C}_i}{\partial t} = - \frac{\partial F_i}{\partial x} + \sum R_i \quad (14)$$

where: $\hat{C}_i$ = concentration of solid or liquid component i in terms of mass per unit volume of total sediment,
 F_i = flux of component i , and
 R_i = reaction rates affecting i .

and

$$F_i = - D \frac{\partial \hat{C}_i}{\partial x} + v \hat{C}_i \quad (15)$$

where: D = diffusion coefficient, and
 v = velocity of flow relative to the sediment-water interface.

If no compaction or infiltration to the groundwater zone occurs, v is the same for the solid and liquid component. Then $v = v_w$ = velocity of burial of solid particles and/or water below the sediment-water interface.

The diffusion coefficient for the solid component near the sediment-water interface (bioturbation and/or current mixing) is D_B . For the dissolved phase, this same D_B plus an irrigation coefficient D_I and a molecular diffusion coefficient D_M which is independent of depth apply.

The concentration of a solid component $\bar{C}_s$ can be expressed in terms of mass per unit mass of total solids

$$\hat{C}_s = (1 - \phi) \rho_s \bar{C}_s \quad (16)$$

and for the dissolved component the concentration C is expressed as mass per unit volume of pore water

$$\hat{C}_d = \phi C \quad (17)$$

Then, combining each of equations (16) and (17) with equations (14) and (15) yields the general equations for the bottom sediments, which for a solid component is

$$\frac{\partial[(1 - \phi) \rho_s \bar{C}_s]}{\partial t} = \frac{\partial[D_B \frac{\partial[(1 - \phi) \rho_s \bar{C}_s]}{\partial x}]}{\partial x} - \frac{\partial[(1 - \phi) v_w \rho_s \bar{C}_s]}{\partial x} + (1 - \phi) \rho_s \Sigma R_s \quad (18)$$

where R_s is the reaction rate in mass per mass of solids per time, and for a dissolved component

$$\frac{\partial(\phi C)}{\partial t} = \frac{\partial[D_B \frac{\partial(\phi C)}{\partial x} + \phi(D_I + D_S) \frac{\partial C}{\partial x}]}{\partial x} - \frac{\partial(\phi v_w C)}{\partial x} + \phi \Sigma R_d \quad (19)$$

where R_d is the reaction rate in mass per volume of pore water per time. For sorbing pesticides, equation (18) can be used for the adsorbed pesticide $\bar{C}$, and equation (19) for the dissolved pesticide. The partition coefficient is used again to describe the equilibrium between the adsorbed and dissolved phases

$$\bar{C} = K_p C \quad (20)$$

where K_p is an average partition coefficient in the bottom sediments (equations (10) and (20) are equivalent).

Furthermore, a mass balance between the two phases must hold, so that the sorption rates R are related to each other by:

$$R_s = \frac{-\phi}{(1 - \phi) \rho_s} R_d \quad (21)$$

From the previous equations the final equation that describes the changes in concentration in the pore water of the bottom sediments is obtained:

$$\frac{\partial C}{\partial t} = \frac{\partial(D_B \frac{\partial C}{\partial x})}{\partial x} + \frac{1}{1 + K'} \frac{\partial[(D_I + D_M) \frac{\partial C}{\partial x}]}{\partial x} - v_w \frac{\partial C}{\partial x} + \left(\frac{1}{1 + K'}\right) (\Sigma R_d + \frac{(1 - \phi)\rho_s}{\phi} \Sigma R_s) \quad (22)$$

where $K' = (\rho_s(1 - \phi) \phi K_p)$.

This equation is solved numerically via finite difference methods with a variable boundary condition, which is the dissolved concentration in the water column. The set of ordinary differential equations describing the water column is solved using the Runge-Kutta method. Because the two systems are interacting with each other they are solved in sequence for each time step.

Results and Discussion

The results of several simulations are shown in the following figures. Typical values for yearly storm runoff data, including runoff volume, sediment mass and pesticide load were used (Smith, et. al., 1977). The sediment size-velocity distributions of the inflow sediments were assumed constant for all storms, and are listed in Table 1. Using the relationships reported by Karickhoff, et. al. (1978) the partition coefficients were estimated based on the organic carbon content of the sediments and a range of typical octanol-water partition coefficients (10^2 - 10^4). In the absence of reported data for mixing coefficients in bottom sediments, reported values for mixing coefficients in near shore silts were used (Bernier, 1980). For the following simulations, a detention capacity of the pond of 2000m^3 and a depth of 2 m was selected.

The bottom sediments were modeled over a depth of 0.2 m. Bioturbation and irrigation was considered to occur only in the upper 10 cm while molecular diffusion was modeled over the entire depth of the bottom sediments. The molecular diffusion coefficient was estimated based on a molecular weight of 200 and corrected for the tortuosity dividing it by the product of the porosity and the formation factor. A formation factor of 3 was assumed.

In Figure 1 the inflow (runoff) pesticide concentration is shown over time, as well as the pesticide concentration in the pond for two different partition coefficients. The effect of the pond can be seen clearly. The peak concentration in the pond is about 3% of the peak storm runoff concentration without a pond. As expected, if the partition coefficient is increased the reduction of the peak concentration is even higher, because more pesticide is in the adsorbed phase, consequently increasing the pesticide removal efficiency of the sedimentation process. For longer times, the effect of the pond is to retain and slowly release pesticides. This effect is increased for higher partition coefficients.

In Figure 2 the total cumulative pesticide load to the receiving waters for consecutive seasons is shown. As expected, if some decay does occur in the water column ($k = 0.02 \text{ day}^{-1}$, for the dissolved phase) the overall pesticide removal rate is increased. It is interesting to notice that for the

case where decay occurs the system reaches its final conditions sooner than if decay does not occur. For the later case, the final conditions would be such that the yearly cumulative loads with and without the pond are the same. The reason for this is that the initial adsorptive capacity of the bottom sediments is exhausted but a buffering capacity still exists. In other words, when the system reaches its final conditions, there is no net gain in pesticide mass in the bottom sediments over a season. This is illustrated in Figure 3, which represents the concentration profiles in the bottom sediments at the end of each year for the cases of Figure 2. The fact that the bottom sediments continue to have a buffering capacity, even if their net yearly gain of pesticides is negligible, is shown in Figure 4. This figure shows the pesticide profile in these sediments at the beginning of the second season, after 50 days, and at the end, for a pesticide with decay.

The effect on the cumulative load of different partition coefficients is shown in Figure 5 in which a pesticide with decay is modeled. The increase in pesticide removal efficiency for increased partition coefficients, during season modeled, results because for higher adsorptivities the influence of sedimentation increases, and also the bottom sediments have a higher capacity to retain pesticides.

For all of the previous simulations it was considered that 54% of the sediments in the inflow are colloidal material or fine clays that will not settle. The improved pesticide removal efficiency when a coagulant is added reducing the non-settling fraction by 75% is shown in Figure 6. These simulations do not include a decay rate. Similar results were obtained for two simulations using low (i.e. fine silt) as well as high (i.e. sand) settling velocities for the coagulated solids.

Summary and Conclusions

The results of these simulations show that sedimentation ponds can be used effectively to dampen out peak pesticide loads to receiving waters. If no decay occurs the total yearly load to the receiving waters will be the same with or without a pond once final conditions (i.e. after a few seasons) are obtained. If some decay occurs, the total yearly load at final conditions will be less with a pond than without one.

Bottom sediments and their interactions with the water column, often ignored for this type of analysis, have been shown to have an important effect on the performance of the pond. It would therefore be extremely valuable to have a better understanding of the mixing conditions in the upper sediment layers of ponds and lakes, as well as the reactions that occur for pesticides and other pollutants in adsorbed and dissolved phases in these sediments.

References

- Berner, R.A., Early Diagenesis, Princeton University Press, Princeton, NJ, 1980.
- Ferrara, R.A., and Salvage, K.M., "Stormwater Pollutant Settleability," in press.

Karickhoff, S.W., Brown, D.S., and Scott, T.A., "Sorption of Hydrophobic Pollutants on Natural Sediments," Water Research, v. 13, pp. 241-248, 1979.

Smith, C.N., et. al., "Transport of Agricultural Chemicals from Small Upland Piedmont Watersheds," USEPA, ORD, Env. Res. Lab., Athens, Georgia, draft report, 1977.

Table 1
Distribution of the Solid Particles in the Runoff

	Particle Size (microns)								
	larger than 150			between 1 and 150			smaller than 1		
% organic carbon	2.6%			5.3%			4.9%		
fraction of the total mass	0.16	0.03	0.01	0.01	0.11	0.04	0.02	0.08	0.54
settling velocity (m/day)	2070	6480	10800	540	1620	2880	195	580	0

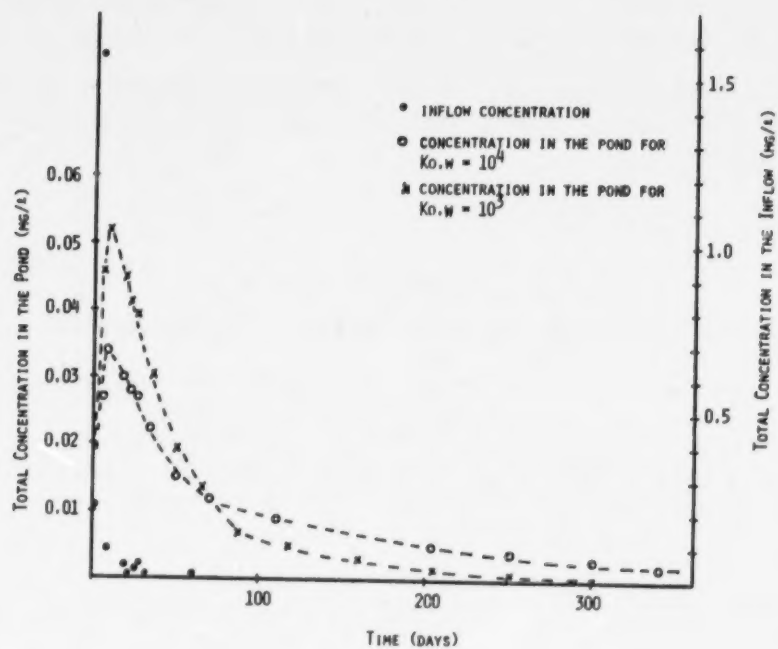


FIGURE 1. ATTENUATION OF THE CONCENTRATION.

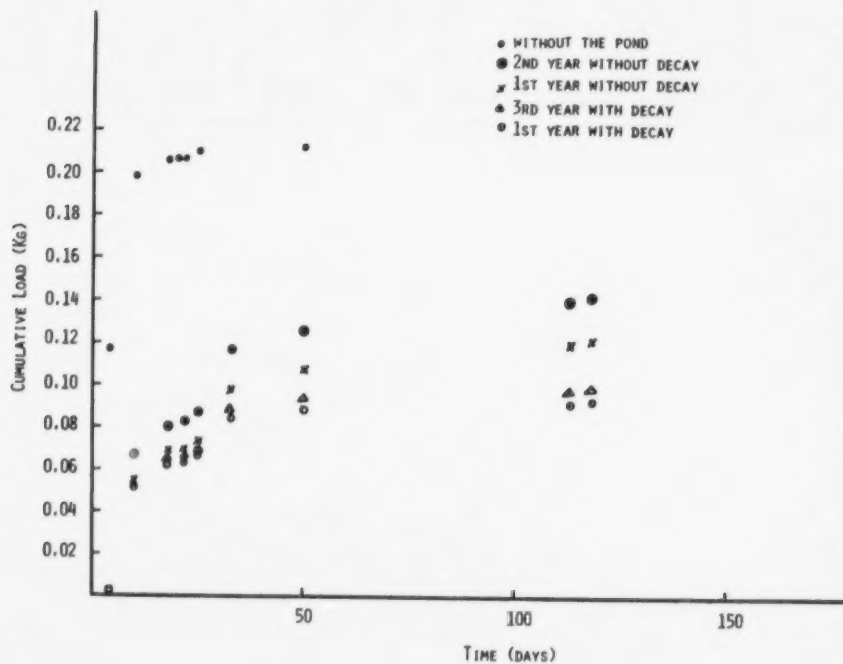


FIGURE 2. CUMULATIVE LOAD FOR CONSECUTIVE SEASONS.

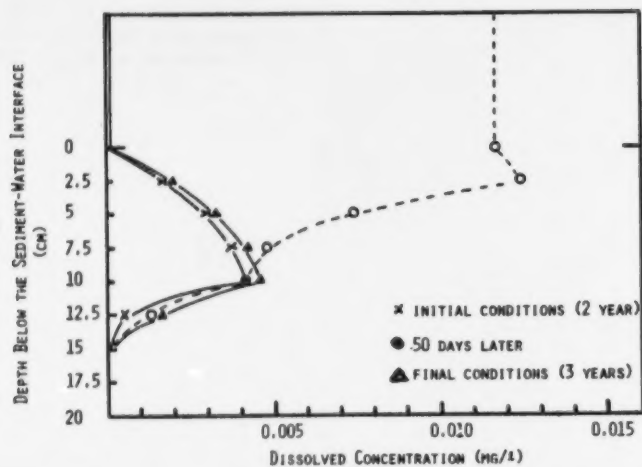


FIGURE 3. PERIODICAL VARIATION OF THE CONCENTRATION IN THE BOTTOM SEDIMENTS.

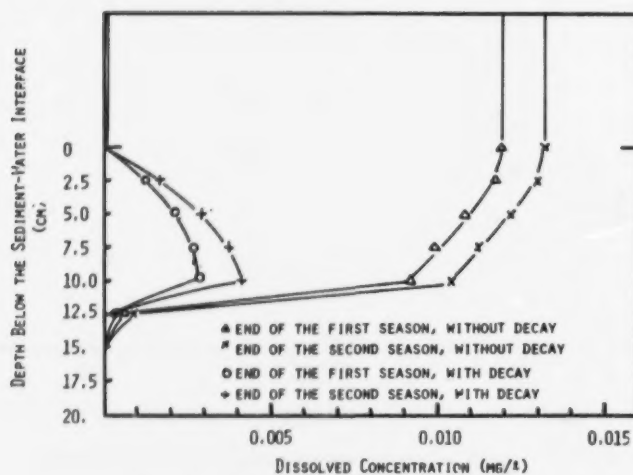


FIGURE 4. EFFECT OF A DECAY RATE ON THE CONCENTRATION IN THE BOTTOM SEDIMENTS.

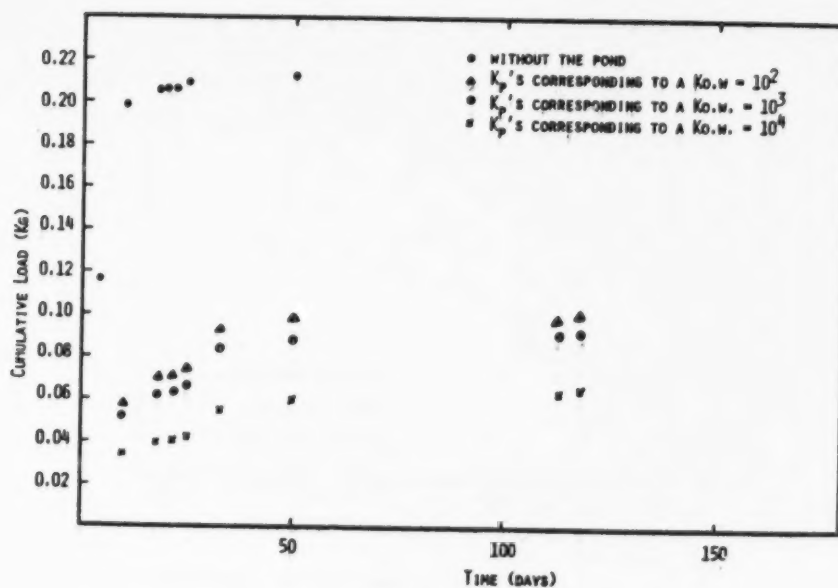


FIGURE 5. EFFECT OF DIFFERENT PARTITION COEFFICIENTS ON THE CUMULATIVE LOAD.

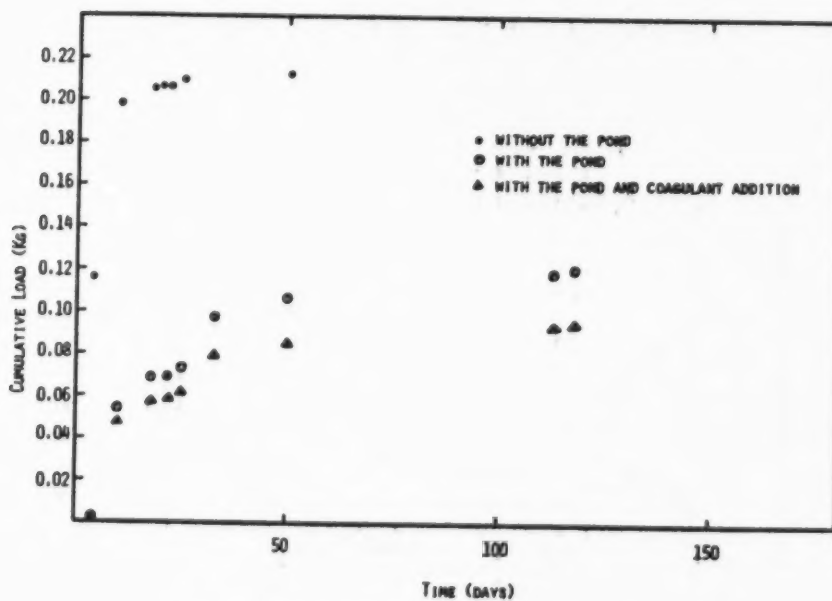


FIGURE 6. EFFECT OF INCREASED SEDIMENT REMOVAL ON THE CUMULATIVE LOAD.

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

A MIXING ZONE MODEL FOR CONSERVATIVE PARAMENERS

by

Main R. Hutcheson
Oklahoma Water Resources Board

SECTION 1 INTRODUCTION

There are two basic ways in which point source discharges are regulated. One way is through the application of technology based permit limits for dischargers. However, the current administration strongly supports the water quality based approach to regulating pollutants (Eidsness, 1982). In this approach a wasteload allocation is performed to determine permit limits which will protect the water quality standards assigned to the receiving stream.

The water quality standards apply everywhere outside the mixing zone. In the past, the thrust has been to ignore mixing zones and perform wasteload allocations for entire planning segments. However, this approach has not proven feasible (Hutcheson, 1979). More states are now including mixing zones. In 1971 twenty-two states mentioned mixing zones, while at present all but three states include mixing zones in their regulations (Neely, 1982). Therefore, a mixing zone model applicable to single discharges or dischargers with overlapping mixing zones can have general applicability.

Mixing zones may be defined in many different ways. Several states use an arbitrary definition involving flow volume, stream width, or distance from the source. Oklahoma defines a mixing zone as being thirteen times the stream width, with three quarters of the flow volume protected as a zone of passage, which effectively limits the mixing zone to about a fourth of the stream width (Figure 1).

In order to assign discharge permit limits (to an industrial point source, for example) which will protect water quality standards outside the mixing zone, it is necessary to employ a dispersion model in the wasteload allocation process. The dispersion model predicts pollutant concentrations in the receiving stream. The most basic and widely used model is the mass balance approach, wherein a steady state system exists and pollutant is mixed uniformly throughout the receiving stream. There are several problems with this approach, the most important being its inability to protect the zone of passage (depicted in Figure 1). Another major problem with the mass

balance approach is verification. Usually the pollutant becomes uniformly distributed within the stream so far downstream that the signal to noise level is too low to relate the effects of the discharge to the instream concentration. Sometimes resuspension, nonpoint sources, partitioning and sedimentation contribute significantly to the concentration of conservative parameters. If the mixing zone is arbitrarily defined, as is the case in many states, then the assumption of uniform mixing through the stream at the end of the mixing zone may be invalid. In Oklahoma complete mixing at the end of the defined mixing zone occurs only on very small streams, so in

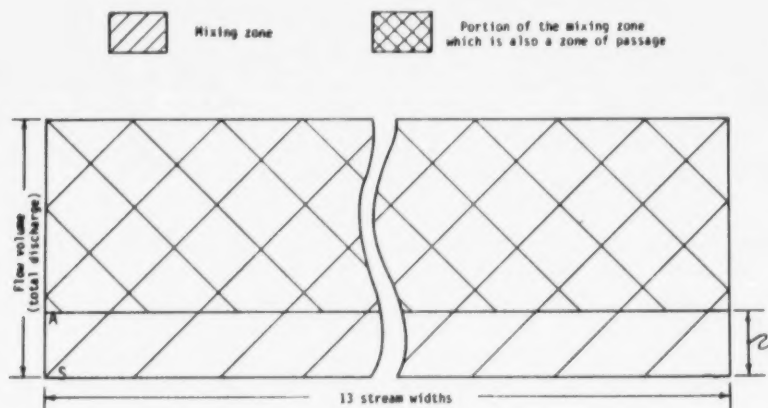


Figure 1 Portion of receiving stream in which water quality standards are not valid for an isolated discharge

general, the concentration outside the mixing zone is much greater than predicted by mass balance (Hutcheson and Gopal, 1981a).

Since dispersion across a stream must be taken into account in the mixing zone, applicable models must include this effect. There are two basic methods for accounting for cross stream dispersion. The easier one assumes that the concentration due to a steady state point source on the banks is half-normally distributed across a stream until reflection from the far bank occurs. Due to the principle of superposition, reflection may be accounted for with a virtual point source. The assumption of a half-normal (Gaussian) concentration distribution is generally reasonable in Oklahoma streams (Hutcheson and Gopal, 1981b). There are many situations where a wasteload allocation methodology based on a Gaussian dispersion model will adequately determine discharge wasteloads (from which permit limits may be derived) which will allow instream concentrations specified by the water quality standards to be met. There are, however, situations where the observed distributions in Oklahoma do not approximate normal ones. Furthermore, it has been shown that the dispersion mechanism is dependent both upon the turbulence of the stream and the configuration of the pollution (Hutcheson,

1979b). In other words, turbulent eddies disperse different sized plumes at differing rates. Therefore, the Gaussian model may not predict concentration distributions as well as a numerical solution to the dispersion equation which computes new dispersion coefficients at various distances from the source does. Furthermore, processes affecting the concentration distribution which depend on the hydrology of the stream are better handled with a numerical solution. An example is partitioning and sedimentation. In general, conservative elements settle out of the water column when the flow speed is slow, not uniformly as would be assumed in a Gaussian model. Therefore, it is deemed worthwhile to pursue a numerical solution to the dispersion equation. This type of model is more amenable to incorporation of other mechanisms affecting pollutant concentration besides dispersion.

The objective of this report is to describe the development of a wasteload allocation process which incorporates a numerical solution to the dispersion equation, and to demonstrate the utility of the process. The development consists of the following steps. First, the dispersion equation is simplified through the use of assumptions so that it may be solved numerically. Then a relationship between the standard deviation of a Gaussian distribution and the dispersion coefficient is derived. Dye studies are used to determine the standard deviations. A relationship for the standard deviation, hence the dispersion coefficient is developed based on hydrology and plume dimensions. The dispersion equation is solved numerically for pollutant concentration at grid points throughout the mixing zone. The maximum concentration at the end of the mixing zone or on the boundary of the zone of passage, the related source strength, the background concentration and the water quality standard are used to determine the wasteload allocation which will protect the water quality standard. This process is applicable to multiple discharges as well as isolated ones, using the superposition principle.

SECTION 2 DERIVATION OF THE APPROPRIATE DISPERSION EQUATION

Dye studies performed in Oklahoma showed that the Natural coordinate system is appropriate for dispersion modeling. The Natural Coordinate system, advocated by Yotsukura and Sayre (1976), features curvilinear horizontal coordinates. The x direction is along the thalweg of the stream, the horizontal coordinate perpendicular to this is the cumulative discharge, q, and the vertical component, y, is stream depth. The cumulative discharge is defined as the discharge from the injection bank to a point in the stream. The maximum q is the total stream discharge.

Hutcheson and Gopal (1981b) showed that, in general, dye concentrations are more like normal distributions when the Natural coordinate system is used rather than a Cartesian coordinate system. Yotsukura and Sayre (1976) also observed this to be the case. Therefore, the Natural system yields better estimates of the dispersion coefficient. Another important reason for using the Natural system in Oklahoma is the definition of the zone of passage. One quarter of the flow volume occurs at the point at which the ratio of cumulative to total discharge is one quarter. Therefore, in a coordinate system using cumulative discharge, the physical dimensions of the zone of passage may be easily determined.

The standard diffusion equation is expressed in the Cartesian system. A coordinate transformation is required to convert to the Natural system, and many assumptions may be made to simplify the equation to facilitate a numerical solution.

In Cartesian coordinates the dispersion equation may be written (Yotsukura and Sayre, 1976)

$$\frac{\partial \theta}{\partial t} + \frac{\partial(\theta w_x)}{\partial x} + \frac{\partial(\theta w_y)}{\partial y} + \frac{\partial(\theta w_z)}{\partial z} = 0 \quad (1)$$

where θ is instantaneous concentration, and w_x , w_y and w_z are flow speeds in the x (downstream along the bank), y (vertical) and z (transverse) directions. Steady state conditions are assumed to exist. Therefore, (1) may be reduced to a three dimensional equation by time averaging (Hutcheson, 1980). The gradient transfer hypothesis (concentration flux proportional to concentration gradient) is assumed valid because the Oklahoma Water Resources Board (OWRB) dye studies showed that the concentration distributions were more or less Gaussian. Further simplification of the dispersion equation requires the assumption of uniform mixing in the vertical. It has been shown that vertical mixing is complete at a distance downstream from the source equal to 50 to 100 times the depth of the discharge point. In shallow Oklahoma streams the assumption of uniform vertical mixing is valid virtually throughout the mixing zone. It is also assumed that the mean velocity is along the thalweg (x direction) and that longitudinal dispersion has negligible influence on steady state mixing. While longitudinal dispersion is the primary dispersion mechanism in non-steady state situations, the longitudinal gradients for the concentration distribution which evolves from a steady state source renders the longitudinal dispersion mechanism ineffective.

Using the above assumptions, dispersion in the q direction and advection in the x direction are the only mechanisms which must be accounted for in order to predict concentration distributions of conservative parameters in a mixing zone. Therefore, employing the appropriate coordinate transformations, (1) becomes (Hutcheson, 1980)

$$\frac{\partial c}{\partial x} = \frac{\partial}{\partial q} (m_x h^2 u K \frac{\partial c}{\partial q}), \quad (2)$$

where c is the time and vertically averaged plume concentrations, h is stream depth, u is stream velocity in the x direction, K is the dispersion coefficient and m_x is a factor introduced to correct for differences between distances along curved coordinate lines and those measured along the x axis. Along the thalweg $m_x = 0$, and at other locations the approximation of Yotsukura and Sayre (1976) may be used:

$$m_x \approx \frac{\Delta L_x}{\Delta x} \quad (3)$$

where ΔL_x is the distance between grid points along a constant q . Eq (2) may be solved if an expression for K can be obtained.

SECTION 3 DETERMINATION OF AN EXPRESSION FOR THE DISPERSION COEFFICIENT K

Dispersion of pollution from a steady state point source is dependent upon the relationship between the characteristics of the prevalent turbulent eddies and the size of the pollutant plume. In order to determine the characteristics of the eddies, perturbation (as opposed to mean) quantities must be measured and assimilated in a statistical fashion. Because of the difficulties in accomplishing this, it was decided to use the relationship between the variance of the concentration distribution and K to determine the dispersion coefficient indirectly. This approach may be used because the Oklahoma dye concentration usually approximated a normal distribution in the Natural coordinate system.

For the conditions under which a normal distribution is valid, h , u and K are independent of q , and $m_x = 1$. Under these conditions (2) may be written

$$\frac{\partial c}{\partial x} - Kh^2u \frac{\partial^2 c}{\partial q^2} = 0. \quad (4)$$

The solution to (4) is the Gaussian plume, expressed by

$$c = \frac{S}{\sqrt{2\pi}\sigma} \exp\left(-\frac{q^2}{2\sigma^2}\right) \quad (5)$$

where σ^2 is the variance of the concentration distribution and S is the source strength. Substitution of (5) into (4) yields

$$K = \frac{1}{2h^2u} \frac{\partial \sigma^2}{\partial x}. \quad (6)$$

This is the diffusivity expression required to satisfy (4) for a Gaussian plume. It will be assumed that it also holds when the concentration distribution is approximately Gaussian.

In order to develop an expression for diffusivity using (6), dye studies were performed on selected Oklahoma streams (Hutcheson and Gopal, 1981b). Ten field experiments were conducted on six different stream reaches. The streams chosen were typical of the larger streams in central Oklahoma. They were broad, flat, shallow streams with smooth sand or mud bottoms containing few rocks or other obstructions. Flow velocities were generally on the order of .5 to 3 feet per second. Dye was injected into the receiving streams at a rate of .2 to .3 ml/second. Samples for dye concentration analysis and flow measurements were taken at frequent enough intervals to adequately define the stream hydrology and plume dimensions to thirteen stream widths from the source.

To determine the variance of the observed dye concentrations symmetry about the injection bank must be assumed since a bank release produces a half-normal distribution. The concentrations observed in the river were placed at the same distance from the injection banks on the inland side to obtain the entire normal distribution. The variance of these concentration distributions were computed using a method employed by Meyer (1977), for equally spaced data

$$\sigma^2 = \frac{\sum [c_i (q_i - \bar{q})^2]}{\sum c_i} \quad (7)$$

where c_i is the normalized steady state concentration at the cumulative discharge q_i and

$$\bar{q} = \frac{\sum c_i q_i}{\sum c_i}.$$

Equally spaced concentrations were extracted from normalized distributions derived from the analyzed samples collected during the dye experiments.

The dye samples were collected on cross sections at various distances from the source. Therefore, the variances computed using (7) are valid for these distances. These computed variances at the cross sections (solid lines) are plotted for seven of the field experiments in Figures 2, 3, 4, 5, 6, 7, and 8 as a function of distance from the source. Data from three experiments could not be used, due to problems with the dye injector on two occasions, and because an injection into the middle of the stream, rather than a bank injection, was attempted once (Hutcheson and Gopal, 1981b).

An examination of Figures 2, 4, 5, 7, and 8 show a tendency for the computed variance to eventually become quasi-constant at some distance from the source. However, since the dye must eventually spread uniformly across the river, the computed variance must continue to increase with distance from the source. Several theories have been advanced for the behavior displayed in these figures, but no definitive answer has been found. The most important behavior displayed in Figures 2-8 is the rapid increase in variance with distance from the source.

Various mechanisms by which the variance increases have been considered. Since only gross features of the flow were measured, it is not possible to relate the dispersion coefficient to turbulence at this time. As shown in Figures 2-8, the variance increases with distance from the source. In order to predict variance using distance, it was assumed that

$$\sigma^2 = ax^b,$$

where a and b are arbitrary constants. The constant values may be obtained using a least squares technique with the computed variances up to the point they become quasi-constant, so

$$\sigma^2 = .0002 x^{2.3}. \quad (8)$$

This equation is used to predict variance, and the results are depicted in Figures 2-8 as the dashed line, so that predicted and computed variances may be compared. The dashed lines are identical in each figure, but appear to shift due to the changing scales.

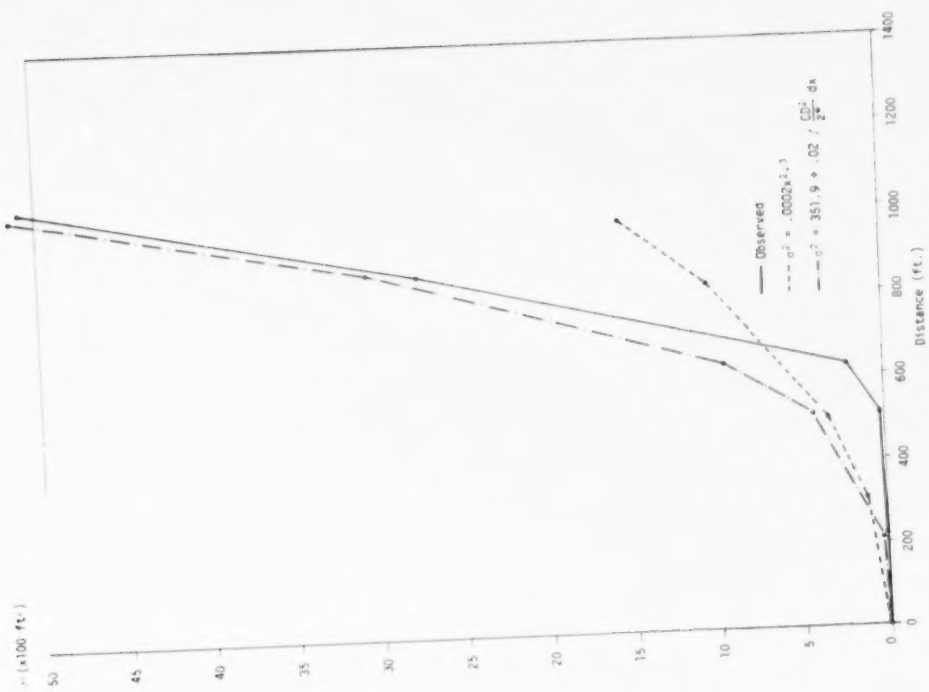


Figure 3 Observed and Predicted Variance versus Distance from Source
State Highway 19

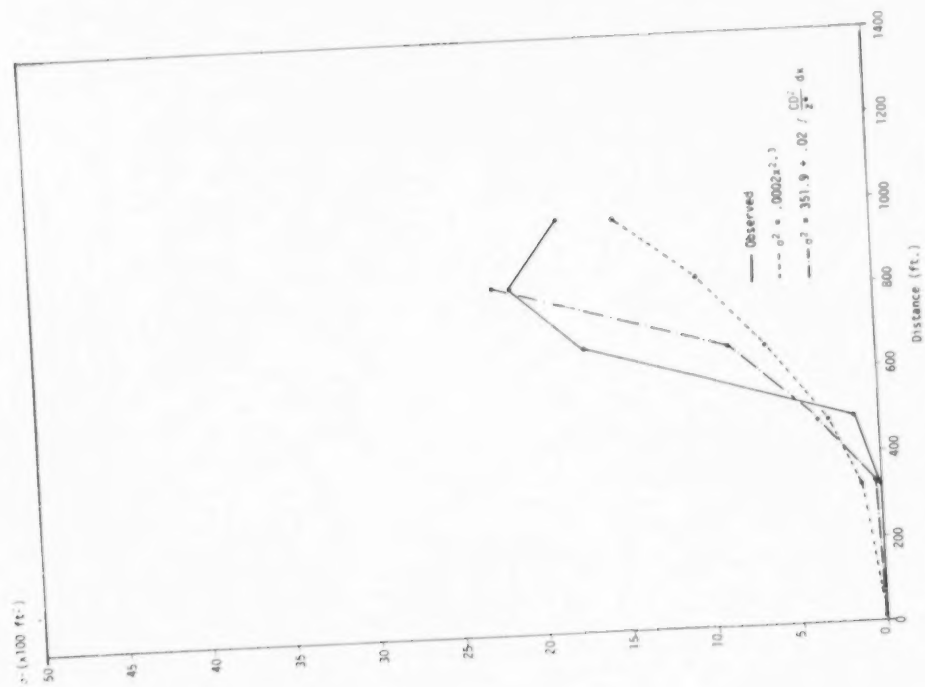


Figure 2 Observed and Predicted Variance versus Distance from Source
State Highway 76

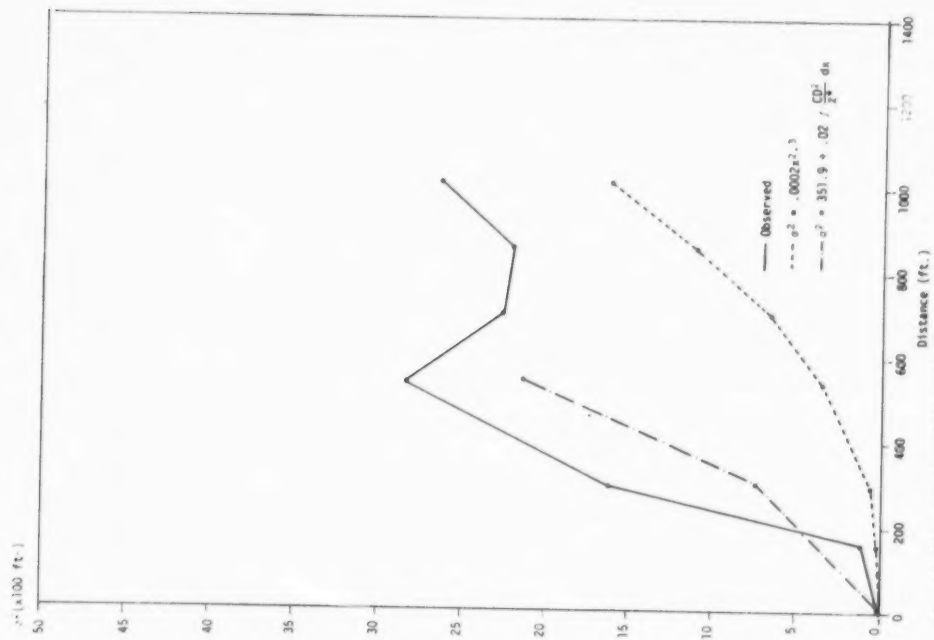


Figure 4 Observed and Predicted Variance versus Distance from Source State Highway 102

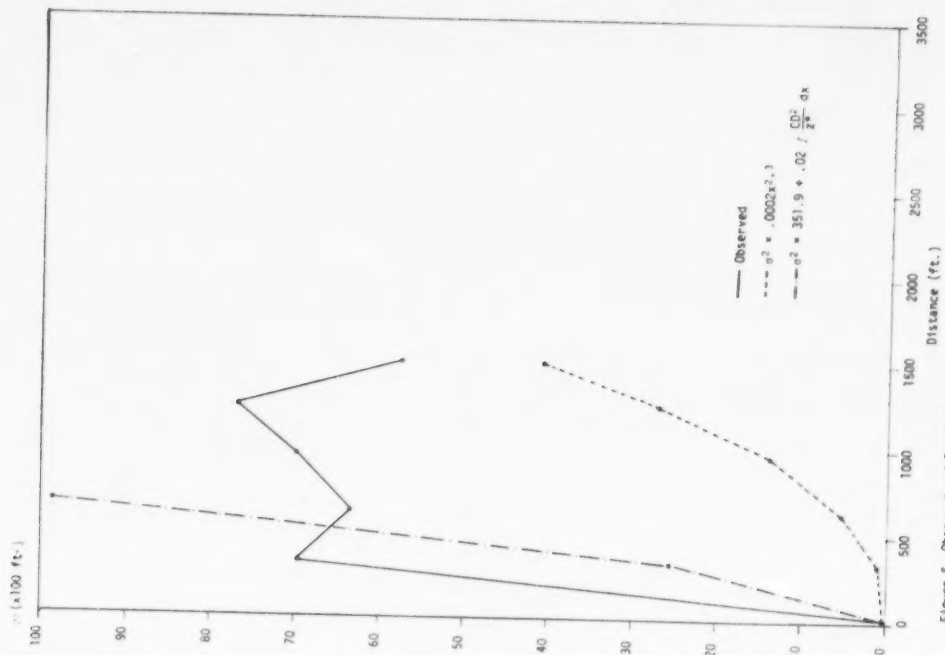


Figure 5 Observed and Predicted Variance versus Distance from Source State Highway 56

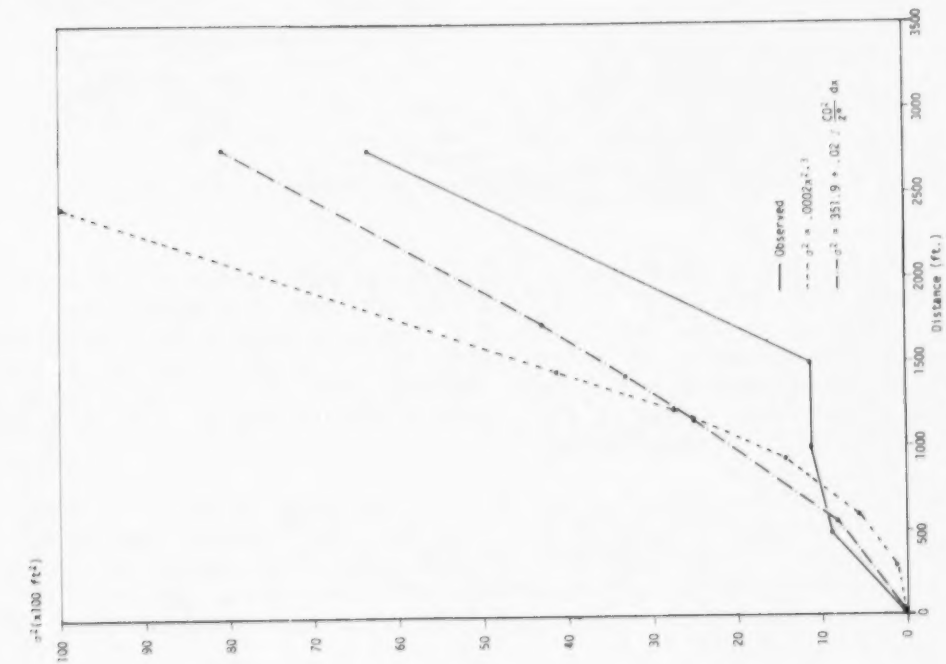


Figure 6 Observed and Predicted Variance versus Distance from Source
State Highway 33

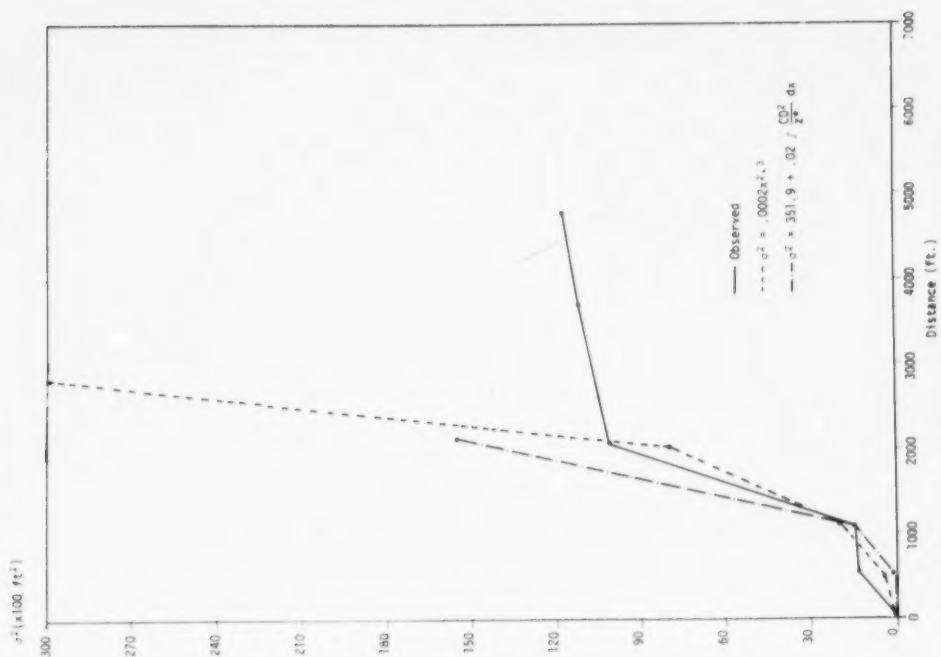


Figure 7 Observed and Predicted Variance versus Distance from Source
U.S. Highway 177A

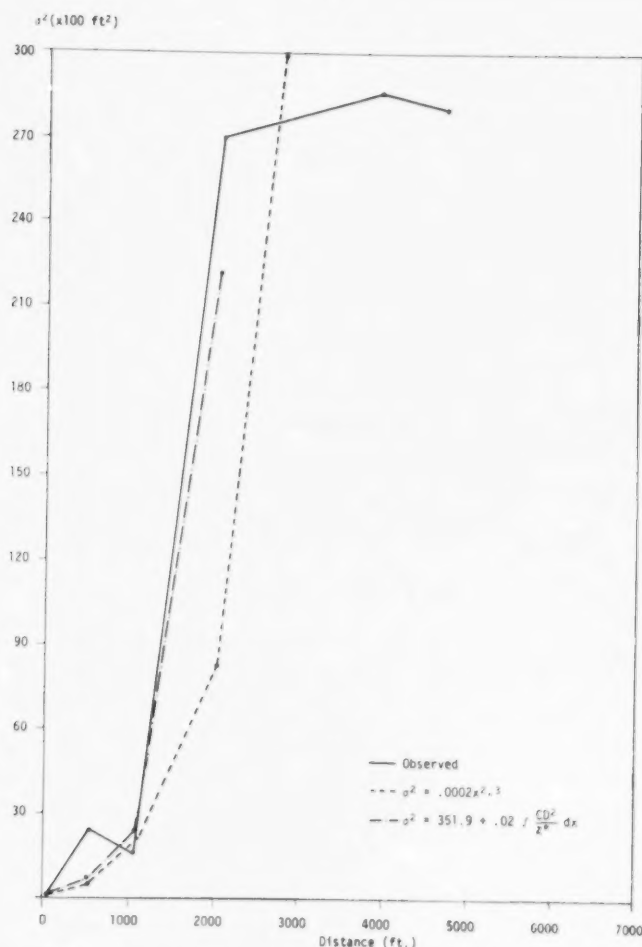


Figure 8 Observed and Predicted Variance versus Distance from Source
U.S. Highway 177B

The correlation between predicted and observed concentration variance is .7. This indicates that (8) does not predict the variance very accurately. Even so, it is tempting to use (8) due to the simple concentration prediction scheme which would result. The expression for σ may be substituted into (5), and, when the mean flow is known, the concentration may be computed as a function of position. This is much simpler than trying to predict concentrations by solving (2) numerically.

It may be shown that, close to the source, the dispersion coefficient changes with distance downstream. A constant coefficient implies σ^2 increases linearly with x , from (6). However, (8) shows the relationship is not linear. Therefore, K is not constant and, therefore, published dispersion coefficients are of little use in predicting concentration distributions near the source.

A relationship between volume of discharge and dispersion was observed during the dye studies. In order to account for upstream dispersion as well

as this relationship, the integral of the cumulative discharge to .3 of the maximum concentration with respect to distance from the source was used as a predictor. The integrated value is used because the variance of the concentration distribution depends upon the dispersion which has occurred at all points downstream from the source. The relationship may be expressed as

$$\sigma^2 = a + b \int \frac{CD}{z^*} dx,$$

where CD is cumulative discharge from the injection banks to the location where the normalized concentration equals .3 and z^* is the distance from the injection bank to this location. The a and b are arbitrary constants found by the least squares technique, so that

$$\sigma^2 = 351.9 + .02 \int \frac{CD}{z^*} dx. \quad (9)$$

Again, the constants were evaluated using only variance and cumulative discharge data up to the point where the increase in variance essentially stopped.

The correlation between the predicted and observed concentration variance is .94. This indicates that (9) is more capable of estimating concentration variance than (8) is. Less than 50% of the variance of σ^2 is explained by (8), while (9) explains nearly 90%. Predictions using (q) are depicted as the dash-dot lines in Figures 2-8. Since the value of the constant in (9) is immaterial, because the expression will be differentiated to obtain K, σ^2 is set equal to 0 at $x = 0$, and = 20 when $\int CD/z^* dx < 200$.

Although (9) is apparently a good predictor for σ^2 , the physical mechanisms governing dispersion remain undefined. Therefore, it is anticipated that much work will be required to adequately predict variance. However, (9) will be used to show the utility of the dispersion equation.

Note that in Figures 2 and 3 there is an initial slow increase in variance, then a rapid increase. In both stream reaches the flow was initially near the bank opposite the dye injection, and it shifted over to the injection bank at the point where the concentration variance started its rapid increase.

In Figures 4 and 5 the rapid increase in variance starts much closer to the source. In these reaches the thalwegs were near the injection banks at the injection points. Predictions using (9) are able to anticipate the point at which the increase in variance starts, while those using (8) cannot.

The same reach is depicted in Figures 7 and 8. Flow conditions are the same, and the only difference is that in Figure 7 the wind is blowing very strongly away from the injection bank, while in Figure 8 the wind is blowing strongly towards the injection bank. This may cause a large enough increase in turbulence near the dye injector to cause the much greater variances observed in Figure 8.

If (9) or a similar expression is used to predict variance, then an analytical expression such as (5) is not appropriate for predicting concen-

tration. The variance must be known in order to use (5) to predict the concentration distribution, but the concentration distribution must be known in order to determine CD and z^* . To resolve this dilemma, the dispersion equation must be solved using numerical methods. This requires the use of an expression for dispersion, rather than one for variance. Since the concentration distributions observed in the dye studies are reasonably normal in the cross-stream direction, (6) should yield a good estimate of the dispersion coefficient. Substitution of (9) into (6) yields

$$K = \frac{.01 CD^2}{h^2 u z^*}. \quad (10)$$

If h and u are constant, then $CD = uhz^*$ and (10) reduces to

$$K = .01 u z^*. \quad (11)$$

Since z^* increases as the plume spread across a river, K must always increase, according to (11). However, K probably becomes constant at some distance from the source, and at this point (10) will no longer be valid. However, the distribution has flattened out so much that, as seen from (4), c changes relatively slowly anyway. Therefore, (10) may be assumed to be valid for thirteen stream widths downstream from the source without unduly degrading the concentration distribution prediction.

SECTION 4 NUMERICAL SOLUTION TO THE DISPERSION EQUATION

Once all of the independent variables have been determined, the equation may be solved for c . Expanding the right hand side (r.h.s.) of (2) and making the appropriate assumptions about K and m_x yields

$$\frac{\partial c}{\partial x} = k \left\{ \frac{\partial^2 c}{\partial q^2} + \left[\frac{\partial(\ln u)}{\partial q} + \frac{\partial(\ln h^2)}{\partial q} \right] \frac{\partial c}{\partial q} \right\}, \quad (12)$$

where $k = m_x h^2 u K$.

Eq (12) implies that K and m_x change slowly enough in the q direction so that their cross stream gradients are negligible in comparison with the gradients of u and h^2 . Certainly the m_x gradient is negligible so long as a stream reach is relatively straight, and the manner in which K is derived requires that it remain constant in the q direction.

In order to solve (12) numerically, it must first be finite differenced. To avoid stability problems, the Crank-Nicholson finite difference scheme is employed. In the x direction, forward differencing is used:

$$\frac{\partial c}{\partial x} = \frac{c(m, n+1) - c(m, n)}{\Delta x}, \quad (13)$$

where Δx is an incremental distance along the thalweg.

The m's indicate locations on the cumulative discharge coordinate (across the stream) and the n's indicate distance downstream along the thalweg.

In the cross-stream direction,

$$\frac{\partial c}{\partial q} = \frac{c(m+1,n+1)-c(m-1,n+1)+c(m+1,n)-c(m-1,n)}{4\Delta q} \quad (14)$$

and similarly for $\ln h^2$ and $\ln u$. The Δq is the incremental discharge in the cross-stream direction. Using the same type of finite differencing:

$$\frac{\partial^2 c}{\partial q^2} = \frac{1}{2(\Delta q)^2} [c(m+1,n)-2c(m,n)+c(m-1,n)+ \\ c(m+1,n+1)-2c(m,n+1)+c(m-1,n+1)] \quad (15)$$

Since $\ln h^2$ and $\ln u$ are known at the grid points, their gradients may be determined a priori. For this reason

$$Y \equiv \frac{1}{2} \left[\frac{\partial(\ln u)}{\partial q} + \frac{\partial(\ln h^2)}{\partial q} \right]. \quad (16)$$

Substituting (13), (14), (15) and (16) into (12), defining $\lambda \equiv k\Delta x/2\Delta q$, and separating the concentration terms involving the n+1 grid points from those involving n yields

$$\lambda \left(\frac{1}{\Delta q} - Y \right) c(m-1,n+1) - (1+Y \frac{2\lambda}{\Delta q}) c(m,n+1) + \lambda \left(\frac{1}{\Delta q} + Y \right) c(m+1,n+1) = \\ \left(\frac{2\lambda}{\Delta q} - 1 \right) c(m,n) - \lambda \left(\frac{1}{\Delta q} + Y \right) c(m+1,n) - \lambda \left(\frac{1}{\Delta q} - Y \right) c(m-1,n) \quad (17)$$

All the quantities in this equation are known except the concentrations at the grid points involving n+1. Therefore, a system of M equations in M unknowns (the concentrations at the n+1 grid points) may be developed.

To solve the system of equations initial and boundary conditions must be specified. The initial condition is required to obtain the concentration at the grid points where $n = 1$. Based upon the dye studies a half-normal concentration distribution located near the source is the appropriate initial condition. A half-normal distribution may be expressed by (5) with S replaced by 2S, to double the concentration (absolute reflection). The concentration variance must be known in order to use (5). Because concentrations were measured in the dye studies and the industry example, the initial variance is computed from the observed concentration. However, a prediction for σ^2 very near the source will be required when concentration measurements are not available. When the concentration at $n = 1$ is determined, (17) may be used in an iterative procedure, so the concentrations at succeeding distances downstream may be determined.

The boundary conditions may be obtained from the anticipated concentration distribution. The half-normal distribution implies very small gradients in concentration at the banks near the source. Far from the source, a uniform concentration distribution again implies very small gradients. Therefore,

the appropriate boundary condition is $\partial c / \partial q = 0$ at the banks. From (14) this is the case if:

$$c(0,n) = c(2,n), \quad c(0,n+1) = c(2,n+1), \quad c(M-1,n) = c(M+1,n), \\ c(M-1,n+1) = c(M+1,n).$$

Expanding (17) into M equations and employing the boundary conditions yields a system of simultaneous equations which may be written in the form of a matrix equation

$$A \vec{C} = \vec{F} \quad (18)$$

where $\vec{C}$ is the column vector

$$\begin{bmatrix} c(1,n+1) \\ \dots\dots\dots \\ c(i,n+1) \\ \dots\dots\dots \\ c(M,n+1) \end{bmatrix}$$

and $\vec{F}$ is the column vector

$$\begin{bmatrix} (\frac{2\lambda}{\Delta q} - 1)c(1,n) - \frac{2\lambda}{\Delta q}c(2,n) \\ \dots\dots\dots \\ (\frac{2\lambda}{\Delta q} - 1)c(i,n-1) + (\frac{1}{\Delta q} + \gamma)c(i+1,n) + (\frac{1}{\Delta q} - \gamma)c(i-1,n) \\ \dots\dots\dots \\ (\frac{2\lambda}{\Delta q} - 1)c(M,n) - \frac{2\lambda}{\Delta q}c(M-1,n) \end{bmatrix}$$

and A is the tridiagonal matrix

$$\begin{bmatrix} -(1 + \frac{2\lambda}{\Delta q}) & \frac{2\lambda}{\Delta q} & 0 & 0 & \dots\dots\dots 0 \\ \dots\dots\dots \\ 0 \dots\dots 0 & \lambda(\frac{1}{\Delta q} - \gamma_i) & -(1 + \frac{2\lambda}{\Delta q}) & \lambda(\frac{1}{\Delta q} + \gamma_i) & 0 \dots\dots 0 \\ \dots\dots\dots \\ 0 \dots\dots\dots 0 & 0 & \frac{2\lambda}{\Delta q} & -(1 + \frac{2\lambda}{\Delta q}) \end{bmatrix}$$

Due to the tridiagonal nature of A, it is possible to invert the matrix and solve for $\tilde{C}$ directly, or an iterative approach may be used.

The mathematical formalism to determine the concentration distribution from a continuous point source located on the bank is now complete. To reiterate, the concentration is first initiated by assuming a Gaussian distribution. Computation of the dispersion coefficient is accomplished in step two using (10). Averaged values of h and u are used, and K is incorporated in the λ 's in the tridiagonal matrix. Concentration at the next grid distance downstream is determined using the last computed concentrations, the newly computed dispersion coefficient, and the required hydrological parameters which are obtained from the observed flow and stream morphology. Step 2 is repeated using the new concentration data, establishing an iterative procedure which is repeated until the concentration at the grid points farthest from the source is determined.

If the input data is totally consistent, then the procedure described will yield a concentration distribution which conserves mass. This, however, is often not the case, and can cause errors in the concentration computations. In order to ensure conservation of mass, a variational formalism is employed (Hutcheson, 1981c).

A numerical solution to the diffusion equation is not new, and a computer program could have been readily obtained from several sources. However, the concept of allowing the model to determine the dispersion coefficient at each grid distance downstream based on the dimensions of the plume and the hydrological characteristics of the portion of the stream affecting dispersion is unique. Therefore, it is deemed more efficient to build the model from basic principles rather than trying to modify an existing computer program.

SECTION 5 MODEL VERIFICATION

Data obtained during the dye studies is used to verify the dispersion model. The dye concentrations were measured on cross sections extending to thirteen stream widths downstream from the source (Hutcheson and Gopal, 1981b). A comparison of the actual dye concentrations with those predicted is given in Figures 9-12. Only the best comparisons are shown here. Eighteen others, some where the comparisons are not nearly so good, are displayed in the Task 111 Final Report (Hutcheson, 1981c). The x's represent the actual dye concentrations and the solid lines represent the predictions. The vertical dashed lines represent the point at which the cumulative discharge equals one-fourth the total discharge, i.e., the edge of the zone of passage. Initialization occurred on cross section A, using the observed concentration variance. The cross sections are essentially equidistant from each other, with A being this distance from the source. The cross sections in Figures 9-12 are 2,770, 2,029, 1,255, and 854 feet from the source, respectively.

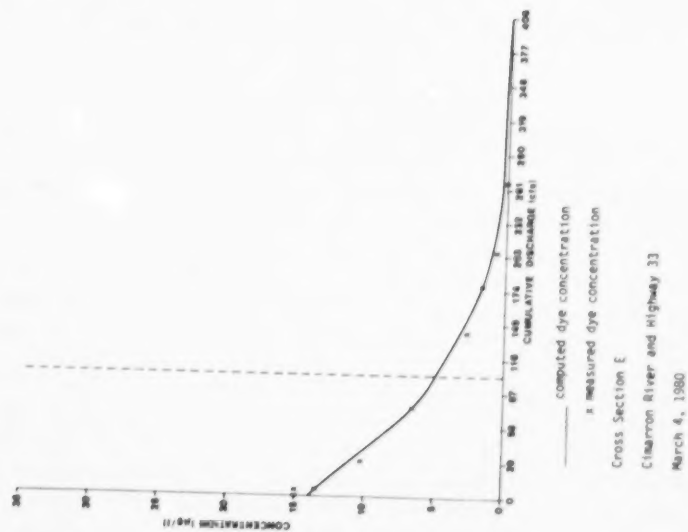


Figure 9 Comparison of computed and measured dye concentrations.

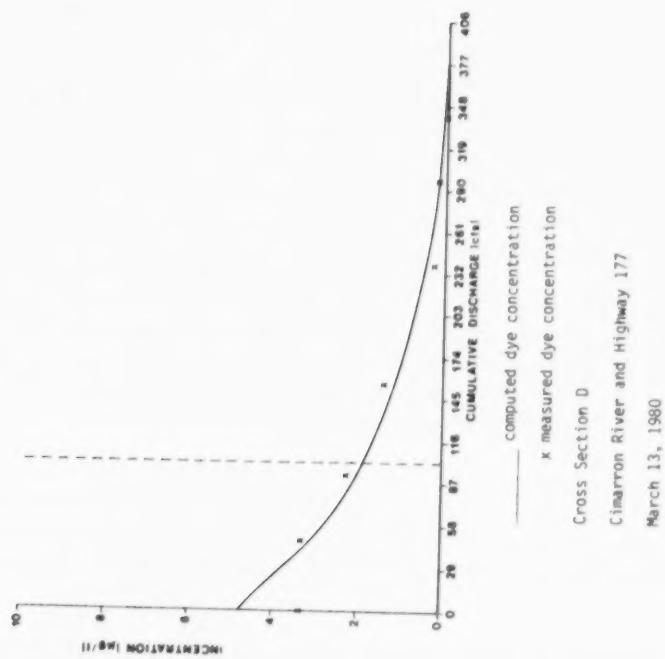


Figure 10 Comparison of computed and measured dye concentrations.

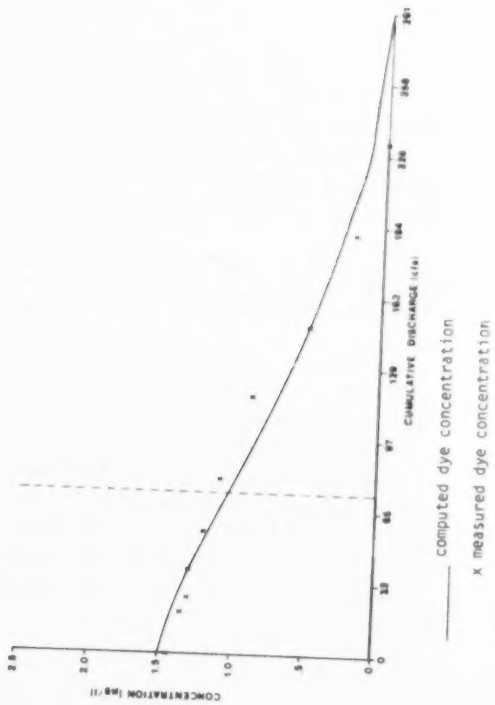


Figure 11 Comparison of computed and measured dye concentrations.

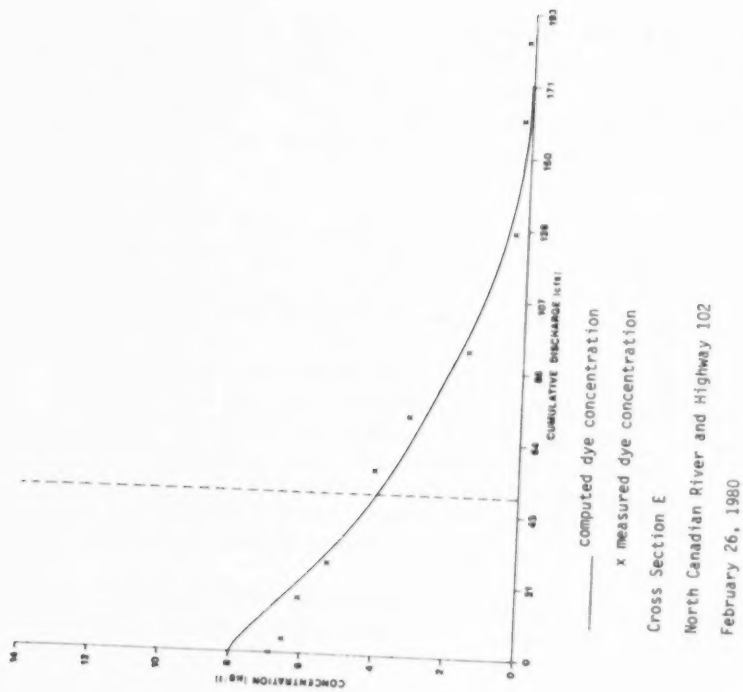


Figure 12 Comparison of computed and measured dye concentrations.

The model adequately reproduces the concentration distributions for the cases presented. This does not mean that in every instance the measured and computed concentrations are identical. In some cases major discrepancies between actual and predicted concentrations exist. More refinement of the model is required before all concentration distributions can be predicted accurately.

When no concentration data is available, initialization must occur close to the source, where the concentration variance is known to be very small. If the plume occupies only a small portion of the discharge, then small grid intervals are required in order to have enough points within the plume to achieve adequate concentration computations. However, smaller grid intervals require more grid points, which require added manual analysis of the data and more entries to the computer. Therefore, it is not feasible to initialize the model on an operational basis unless an objective analysis scheme is developed to alleviate the manual analysis and data entry problem. Since the initial variances measured in the two stream reaches examined on the Washita River are very small, a finer grid mesh than is employed by the dispersion model is required. For this reason, these cases were not run.

It is very difficult to make the input data compatible with the dispersion model using a manual analysis. The amount of time required to manually adjust the flow velocity, depth, and total discharge on a grid dense enough for use in the dispersion model is prohibitive, especially since m_x must also be accounted for. Therefore, an objective analysis scheme which ensures compatibility between the model and input data must be developed to ensure conservation of mass.

In order to make the model valid for all situations, a better dispersion coefficient is required. The K currently used in the model is not related closely enough to the turbulence of the flow to be useful in all situations. For example, the effect of bottom roughness on turbulence is not taken into account. Therefore, the current dispersion coefficient cannot be valid except for the smooth sand or mud bottoms on which it was developed.

SECTION 6 WASTELOAD ALLOCATION METHODOLOGY

The wasteload allocation technique uses the assimilation capacity of the receiving stream as a basis for permitting. This is accomplished through the use of a dispersion model. It is recognized, however, that technology based permit limits and socio-economic considerations are also vital parts of the wasteload allocation process. Even when the assimilation capacity is virtually unlimited, dischargers should be required to meet some minimal level of treatment, such as best practicable treatment. In some cases the assimilation capacity is so small that it is impossible to achieve stringent water quality standards, so socio-economic factors come into play. These factors are not considered here, however.

To obtain wasteload allocations, concentrations generated by the dispersion model must be used. Because the principle of superposition applies to conservative parameters, it is easy to extend the dispersion model, developed to predict concentrations for a single source, to multiple discharges. Therefore, the model may be run for each discharger, and the results added to determine total concentration at the grid points.

The wasteload allocation process is designed to use the model results to determine permit limits which restrict discharges so that concentrations on the boundary do not exceed standards. First the background concentration must be estimated. This may be done in any one of several ways (Hutcheson, 1982). Next, concentrations on the boundary formed by the mixing zone and the zone of passage, as shown in Figure 1 are determined using the dispersion model. For wasteload allocation purposes, any reasonable source strength, S_1 , may be used. However, for multiple discharges the source strengths must be related to each other so that

$$S_i = \alpha_i S_1 \quad 2 \leq i \leq n, \quad (19)$$

where the α 's are determined a priori and there are N discharges of a conservative parameter within the composite mixing zone described by the Oklahoma Water Quality Standards (1982).

The concentrations computed on the boundary are examined to determine the maximum, C. In larger streams the maximum concentration is usually found at the end of the mixing zone, while in very small streams it's likely to be at the edge of the zone of passage. In order to ensure that the water quality standards are met everywhere on the boundary, a source strength, S'_1 , is required to produce the concentration C' specified by the water quality standards. It may be shown (Hutcheson, 1982) that the required source strength can be determined from

$$S'_1 = \frac{C' - B}{C - B} S_1. \quad (20)$$

S'_1 is the permit limit if technology based permit limits or socio-economic factors do not enter into the wasteload allocation process.

Once S'_1 is determined, the remainder of the required source strengths may be obtained from (19). Several combinations of α 's must be employed in order to determine which of these yields the largest allowable wasteload. None of the sources should be allowed to discharge in excess of the appropriate technology based limits.

The procedure outlined above allows for total allocation of the assimilative capacity of a receiving stream. Once the stream is fully allocated, additional or increased discharges can be allowed only when accommodation is made so that water quality standards are not violated.

SECTION 7
APPLICATION OF THE WASTELOAD ALLOCATION METHODOLOGY
TO AN INDUSTRIAL POINT SOURCE

Farmland Industries discharges into Skeleton Creek. Velocity, depth, and water samples were taken at appropriate locations in the mixing zone (Hutcheson and Gopal, 1981a). The water samples were analyzed for zinc, chrome, and total dissolved solids (TDS). Farmland Industries does not discharge continuously. However, its period of discharge is assumed long enough for a steady state situation to be established within the mixing zone. Therefore, the wasteload allocation methodology may be applied.

Even though wasteload allocations for three different parameters were performed, it was necessary to run the dispersion model only once to obtain C for a given S_1 (Hutcheson, 1982). The background concentrations were obtained from analyzed samples and C' for each parameter from the Oklahoma Water Quality Standards. These standards have since been revised. Eq. (20) was used to obtain the source strength (proposed permit limit) for the three parameters.

The wasteload allocations for Farmland Industries can be made without the use of the dispersion model, since the maximum concentrations on the boundary are known from an analysis of the samples collected. Therefore, the observed concentrations and observed source strengths may be used in (20) to determine allowable wasteloads. A comparison of the proposed allowable loads is shown in Table 1.

Table 1. Comparison of wasteload allocations.

C-B (mg/L)	S_1 (mg/sec.)	Wasteload Using Observed Concentration (mg/sec.)	Wasteload Using Dispersion Model (mg/sec.)
.2435	28.664	<u>Chromium</u> 4.12	6.83
.2745	27.56	<u>Zinc</u> 499.4	962.1
1619	524,872	<u>TDS</u> 116,415	69,456.5

Wasteloads using the water samples are shown in column three, while those obtained using the dispersion model are shown in column four. The first two columns show the difference between the observed maximum concentration on the boundary and the background, and the observed source strength,

respectively. For chrome and zinc, the allowable wasteload produced using observed concentrations is smaller than that using the dispersion model. However, for TDS the allowable wasteload produced using observed concentrations is larger.

While there are discrepancies between the wasteload allocations, they are relatively insignificant. A more important comparison is between the wasteloads and column 2, which represents the actual wasteload being discharged at the time of observation. The allowable wasteload (represented by either column 3 or 4) for chromium and TDS is much lower than the actual wasteload. Therefore, regardless of which method of computing maximum allowable wasteload is used, a great reduction in the observed wasteload is required to meet the water quality standards which were applied. However, the allowable wasteload for zinc is much larger than the observed wasteload. Therefore, zinc loading could be increased without violating water quality standards.

SECTION 8 CONCLUSIONS

A wasteload allocation methodology has been developed. It does not rely on the assumption of a uniform concentration distribution, and can be applied to either isolated or multiple outfalls. The methodology provides a means for determining the wasteload allocation required to meet water quality standards, thereby protecting the beneficial uses assigned to a receiving stream. The central component of the methodology is a dispersion model for the mixing zone for conservative parameter discharges.

There are several improvements which should be made to the dispersion model in order for it to be used on an operational basis in the wasteload allocation procedure. Input to the dispersion model is much too tedious for operational use, so an objective analysis must be developed to minimize manipulation of data to make it more compatible with the dispersion model. The initialization procedure is too dependent upon the judgement of the person operating the model; it must be made more automatic. The determination of the dispersion coefficient is not based on physical principles, so more data should be gathered and a coefficient which explains the dispersion mechanism more fully should be developed. Other mechanisms which affect concentration, such as settling, must also be incorporated into the model.

There are many situations in which the wasteload allocation described is not applicable. Because it is based upon a steady state mixing zone model for conservative parameters, it cannot be used if the discharge interval is too short to allow steady state conditions to develop in the mixing zone, if the pollutant is not "conservative," or if a classical mixing zone is not appropriate for the receiving water body. This occurs when there is no flow velocity, as in a lake, or when the discharge forms a majority of the flow.

However, even in its present form, the utility of the wasteload allocation methodology has been shown. It has been demonstrated that the dispersion model can predict concentration distributions well enough to develop permit limits which will allow water quality standards in a receiving stream to be met.

REFERENCES

- Eidsness, Frederic A. 1982. Prepared Text on Water Quality Standards. Presented at the Governor's Water Conference, Oklahoma City.
- Hutcheson, Main R. 1979a. Select and Verify Hydrologic Model(s) for Conservative Parameters. Final Report, 208 Work Element No. 613, Oklahoma Water Resources Board.
- Hutcheson, Main R. 1979b. Analysis of Ensemble Averaged Concentrations and Fluxes in a Tracer Puff. EPA-600/4-79-002, National Technical Information Service, Springfield, VA.
- Hutcheson, Main R. 1980. Refine and Update Model for Conservative Parameters. Working Paper, 208 Work Element 111. Oklahoma Water Resources Board.
- Hutcheson, Main R. and B. K. Gopal. 1981a. Mixing Zone Survey. Final Report, 208 Work Element No. 245, Oklahoma Water Resources Board.
- Hutcheson, Main R. and B. K. Gopal. 1981b. Model Calibration and Verification Studies for Conservative Parameters. Final Report, 208 Work Element 232. Oklahoma Water Resources Board.
- Hutcheson, Main R. 1981c. Refine and Update Model for Conservative Parameters. Final Report, 208 Work Element 111. Oklahoma Water Resources Board.
- Hutcheson, Main R. 1982. Develop a Wasteload Allocation Methodology and Determine Maximum Allowable Loads for Conservative Parameters. Final Report, 208 Work Element 121. Oklahoma Water Resources Board.
- Meyer, W. 1977. Transverse Mixing in the Mobile River, Alabama. Journal Research, U.S. Geological Survey, Volume 5, No. 1.
- Neely, W. Brock. 1982. The Definition and Use of Mixing Zones. Environ. Sci. Technol. Vol. 16, No. 9, pp. 518-521.
- Oklahoma Water Resources Board. Oklahoma Water Quality Standards (1982).
- Yotsukura, Nobuhiro and William W. Sayre. 1976. Transverse Mixing in Natural Channels. Water Resources Research, Vol. 12, No. 4, 695-704.

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

SOME RECENT ADAPTATIONS AND APPLICATIONS OF QUAL-II IN THE NORTHEAST

presented at the

Stormwater and Water Quality Model Users Group Meeting
University of Florida, Gainesville, January 27-28, 1983

by

William W. Walker, Jr., Environmental Engineer
1127 Lowell Road, Concord, Massachusetts 01742

Introduction

The QUAL-II model has an extensive history which can be traced to the original work of Streeter and Phelps (1925) and has been widely used in wasteload allocations and other aspects of river basin water quality management. Several versions of the program have appeared (Texas Water Development Board, 1970, Water Resources Engineers, 1972, Meta Systems, 1979, Roesner et al., 1981). Generally, all versions provide a capability for simulating longitudinal transport and transformation of water quality components in one-dimensional, vertically-mixed systems with steady-state hydraulic conditions. This paper describes certain modifications which have been made in the model structure to improve simulations of dissolved oxygen conditions in rivers heavily impacted by photosynthesis. The code has also been adapted for use on microcomputers. The development of this version is traced to wasteload allocation studies in Vermont (Meta Systems, 1979, Vermont Department of Water Resources, 1982) and to recent studies in Maine (Walker, 1982) and Massachusetts (Walker, 1983).

Model Modifications

The simulation of nutrient cycles and algal growth kinetics tend to be more important in rivers and shallow impoundments with low velocities. In these situations, photosynthesis by suspended phytoplankton, aquatic plants, and/or periphyton may represent important components of the oxygen balance. Several alterations have been made in the QUAL-II structure to improve simulations of phytoplankton, nutrients, and oxygen under these conditions. These include:

- (1) addition of detrital organic phosphorus and organic nitrogen compartments;
- (2) provision for algal uptake of ammonia and/or nitrate nitrogen (vs. nitrate only in previous versions);
- (3) provision for self-shading by phytoplankton; (computing light extinction coefficients as a function of chlorophyll introduces an important feedback control on peak biomass levels in nutrient-rich environments);

- (4) specification of alternative (vs. multiplicative) nutrient limitation by nitrogen or phosphorus;

Control pathways are depicted in Figure 1. The addition of detrital organic nutrient compartments essentially closes the nutrient cycles and permits model calibration and testing against observed total nitrogen and total phosphorus data, as well as individual nutrient species. The other modifications are designed to reflect the kinetic formulations used in state-of-the-art phytoplankton models, as applied to lakes and estuaries (Di Torro et al., 1977). Details on the equations and functional forms are given elsewhere (Vermont Department of Water Resources, 1982, VanBenschoten and Walker, 1982).

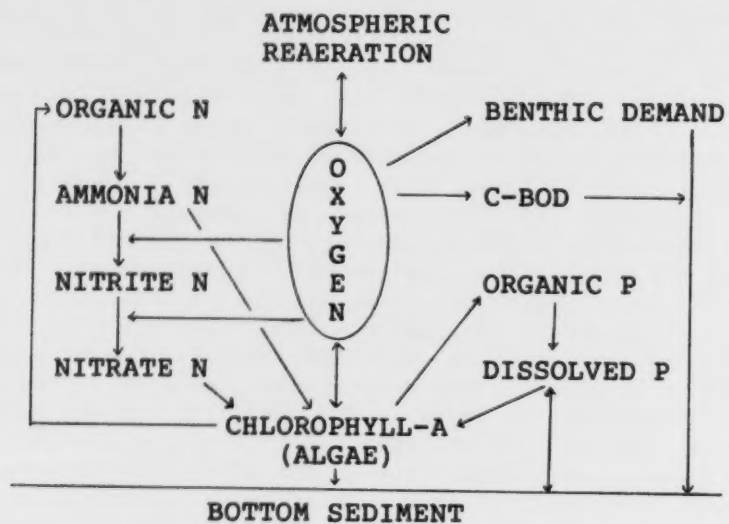
Other additions and structural modifications used in the applications discussed below include:

- (1) calculation "Apparent BOD-5" concentrations as a function of carbonaceous BOD and estimated 5-day algal respiration at 20 deg C (permits calibration and testing against observed BOD-5 data);
- (2) calculation of Secchi depths from chlorophyll and non-algal light extinction coefficients (also useful in calibration and impact assessments);
- (3) output of several diagnostic variables useful for identifying controlling processes, including breakdowns of oxygen sources and sinks ($\text{g/m}^3\text{-day}$) and algal growth factors in each computational element;
- (4) provision for specifying benthic sources and/or sinks for any state variable by reach in $\text{g/m}^2\text{-day}$, including plant photosynthesis and respiration;
- (5) provision for including dam reaeration at the downstream end of any reach;
- (6) provision for simulating eddy-diffusive exchanges with downstream water bodies by specifying far-field concentrations and effective dispersion rate (useful for simulating fresh-water estuaries or other backwater situations where the downstream water quality boundary condition is fixed);
- (7) provision for estimating longitudinal dispersion coefficients using a function developed by Fischer et al. (1979), reportedly more realistic than the Elder (1959) equation;
- (8) provision for simulating diel oxygen fluctuations attributed to photosynthesis and respiration using an approximate formulation described by DiTorro (1968);

The last modification permits calculation of daily minimum oxygen concentrations without applying the model in a non-steady-state mode. Diel fluctuations are calculated around the steady-state solution as a function of

Figure 1

Control Pathways in the Modified QUAL-II Model



reaeration rate, photosynthesis rate, and day length.

In the Vermont application, most of the above changes have been implemented by modifying certain subroutines in the EPA code and adding new subroutines for simulating detrital organic nutrients and diel oxygen variations. A SAS (SAS Institute, 1979) interface has also been developed for manipulating model output and plotting observed and predicted concentration profiles (Walker, 1980). The interface has been extremely useful in streamlining calibration, testing, and report writing.

In subsequent applications, the program code has been completely rewritten and adapted for use on a microcomputer. The micro version is written in standard FORTRAN and uses overlays to overcome storage limitations. It has also been run on mainframes. Experimentation indicated that a version written in interpreted BASIC would be too slow and storage-limited to be practical in most applications. Application limits are determined primarily by the number of computational elements, computer memory, and the memory overhead consumed by the FORTRAN compiler, linker, and input/output operations. On a system with approximately 40K of available storage (64K less overhead), the current version can handle up to 190 computational elements. Provisions for branching or non-steady-state simulations are not included, primarily because they have not been required in applications to date. These restrictions could be lifted with additional code modifications and overlaying. The micro version has been tested against the modified EPA code using input files developed for the Vermont Winooski River application. A separate program has also been written for plotting observed and predicted profiles. Execution times for a typical problem, including printing and plotting, are on the order of ten minutes on an 8-bit, 4 mhz microcomputer equipped with a special arithmetic chip available for most systems.

Upgrading of the algal growth kinetics (especially the inclusion of self-shading) reduces the linearity of the equations has been found to pose problems for the steady-state solution algorithm, particularly in systems with long hydraulic residence times. In the current EPA code (Roesner et al., 1981), convergence is guided and tested based upon algal growth rates. In the modified code, convergence is tested based upon stability of the state variables from one iteration to the next. Certain modifications of the tri-diagonal matrix formulation have also been successful in improving convergence properties, especially in systems with long residence times or significant algal growth limitation by self-shading or nutrients.

Calibration and Testing Procedures

Model inputs may be broadly classified as (1) "boundary conditions" or (2) "system parameters". Boundary conditions include such factors as river flows, waste inputs, morphometry, and climate; these can be directly measured or independently estimated. System parameters include the fundamental rate and stoichiometric coefficients which are used in process-level simulations and which are usually difficult to measure directly. In simulating a given river system, the boundary conditions vary from one time period to another, depending upon various driving forces, while the system parameters should be relatively constant.

The development of input estimates within each category is based upon combinations of the following:

- (1) direct monitoring data
- (2) empirical functions (e.g. reaeration rate formulae)
- (3) literature values and guidance manuals (e.g., Zison et al., 1978, NACASI, 1980)
- (4) empirical adjustment ("tuning") to observed water quality profiles
- (5) field reconnaissance

The calibration process would be relatively straight-forward if all inputs could be directly measured. This is generally infeasible, however, because of the complexity of the model, implicit nature of some coefficients, and limitations in monitoring resources and technology. Calibration of this type of model requires subjective judgments because the feasible ranges of most coefficients are wide and more than one set of coefficients can often be selected to fit a given set of field data. Because of these limitations, user experience and field reconnaissance are valuable assets in model applications. Sensitivity analysis should be employed to determine the importance of parametric and structural assumptions.

One test of model generality is based upon stability of the process-level parameters from one time period to another in a given river. This test involves simulating observed water quality conditions under at least two different sets of boundary conditions using a fixed set of parameters. A more severe test of generality would require parameter stability from one river to another. A model with this property, while difficult to achieve, would be extremely useful because it would ease the calibration and testing requirements for each application.

Because of the desirability of achieving generality and learning more about the strengths and weaknesses of the model structure, a relatively conservative approach involving minimal adjustment of system parameters has been taken in calibrating the model to the systems described below. Most parameter adjustments have been limited to characteristics which could logically vary from one river to another, such as benthic photosynthesis or benthic oxygen demand; these are essentially forcing functions, as opposed to system parameters. Stoichiometric coefficients (e.g., respiration equivalents, algae chlorophyll-a and nutrient contents) and many rate coefficients have been held constant at "reasonable" values, based upon literature ranges and accumulated experience with the model.

Many parameters have been estimated with the aid of published empirical relationships (e.g. reaeration rate), rather than adjusted to fit individual profiles. Direct field measurements of reaeration rate would be preferable, but are usually infeasible because of time or economic constraints. Variations in certain process-level parameters, such as algal growth rate, respiration rate; and settling velocity, from one reach to another in a given river would introduce too many degrees of freedom in the calibration process and are less defensible on a scientific basis; accordingly, reach-to-reach adjustments in these parameters are not part of the calibration procedure.

Applications

The model has been applied in studies of three river basins in the Northeast: (1) Lower Winooski River, Vermont (VanBenschoten and Walker, 1982); (2) Upper East Branch of the Sebasticook River, Maine (Walker, 1982); and (3) Sudbury/Concord Rivers, Massachusetts (Walker, 1983). While there are several unique features associated with each of these applications, all include low-velocity reaches in shallow impoundments or backwater areas with high concentrations of algae (generally greater than 30 mg chlorophyll-a/m³) during critical periods. Key parameter estimates are summarized in Table 1. Model generality is reflected by the stabilities of some parameters and instabilities of others from one system to another. Each application is described briefly below.

The Winooski River originates in the Green Mountains of Vermont and empties into Lake Champlain. In its last 32 kilometers, the river flows through the metropolitan Burlington area, where there are two hydropower dams and several industrial and municipal point sources. Violations of the 6 mg/liter dissolved oxygen standard have generally been observed near the mouth of the river, where the elevation gradient is relatively small and where maximum summer algal populations of about 30 mg/m³ are generally found. The oxygen violations are usually associated with large diel fluctuations (up to 8 mg/liter), attributed to combined effects of (1) algal photosynthesis and respiration and (2) daily fluctuations in river flow resulting from hydropower operations (typically 50 to 1200 cfs). Sustained periods of oxygen violations have also been observed at various locations under conditions of maximum temperature (> 30 degrees C) and die-off of algal blooms.

The Vermont Agency of Environmental Conservation (VAEC, 1980, 1982) conducted a wasteload allocation study in order to assess possible requirements for advanced waste treatment. The unsteady flow regimes induced by hydropower peaking operations posed several potential problems for the modeling effort. Because of the complexities and extensive data requirements involved in application of a non-steady-state hydraulic and water quality model, the Agency elected to use a steady-state model and to conduct the allocation runs under steady 7Q10 conditions, under the assumption that the utilities would be required to pass a minimum of 7Q10 during non-generating hours. The model described above was calibrated and tested against data from week-long intensive surveys conducted during two different years. With the cooperation of the power company, stable flow conditions were maintained during both surveys to provide suitable data sets for calibration and testing of a steady-state model. Observed and predicted concentration profiles for the second survey are shown in Figure 2.

The Sebasticook study (Walker, 1982) examined the impacts of a combined municipal/industrial discharge on a small stream which discharges into an inlet of Lake Sebasticook, a eutrophic lake in central Maine. The shallow inlet is characterized by high algal and aquatic plant densities and hydraulic exchanges with open lake waters are probably important in determining concentrations at the lower end. Preliminary model simulations indicate that the inlet is functioning as an oxidation pond, since oxygen turnover rates in the water column consist largely of photosynthesis and respiration by algae and aquatic plants. A detailed evaluation is hindered by lack of diurnal sampling for dissolved oxygen and limited spatial and

Table 1
Parameter Estimates Used in Winooski, Seabasticook, and Sudbury Applications

Parameters	Value/Comments
Reaeration Rate	O'Connor & Dobbins (1958) equation constrained to $K_2 > 1$ / mean depth (Su)
Longitudinal Dispersion	Elder(1959) (Wi), Fischer (1979) (Se,Su)
Decay/Oxidation Rates (1/day)	
BOD-U	.2 + bed activity (Zison et al., 1978)
Ammonia N	.2 - 2 (Wi), .3 - 1 (Se), .6 (Su)
Nitrite N	2.0 (Wi,Se), 3.0 (Su)
Organic N	.1
Organic P	.1
Algal Parameters	
Maximum Growth Rate	2.3 1/day (Wi,Su), 2.5 1/day (Se)
Respiration Rate	.12 1/day
Settling Velocity	.75 m/day (Wi,Su), .60 m/day (Se)
Chlorophyll Content	.010 mg Chl-a / mg Algae
P Content	.011 mg P / mg Algae
N Content	.080 mg N / mg Algae
Light Extinction	43.2 m ² /g Chl-a
Ammonia Pref. Factor	.9
Photo. Oxygen Equiv.	1.6 mg O ₂ / mg algae
Resp. Oxygen Equiv.	2.0 mg O ₂ / mg algae
Half-Saturation Constants	
Algal P Uptake	.005 g/m ³
Algal N Uptake	.03 g/m ³
Algal Growth vs. Light	1.5 calories/cm ² -hr
Benthic Oxygen Fluxes (g/m ² -day)	
Plant Photosynthesis	0 - 2 (Wi), 0 - 2 (Se), 1.5 - 10 (Su)
Plant Respiration	0 (Wi), 0 - 2 (Se), 1.5 - 10 (Su)
Other Benthic Demand	.5 (Wi), .5 (Se), 1 - 3 (Su)*
Other Benthic Fluxes (g/m ² -day)	
Dissolved P Source **	0 - .005 (Se,Su)
Ammonia N Source **	0 - .025 (Su, channel)
Ammonia N Sink	.05 (Su, overbank)
Nitrate N Sink	.10 (Su, overbank)

Wi = Winooski, Se = Seabasticook, Su = Sudbury

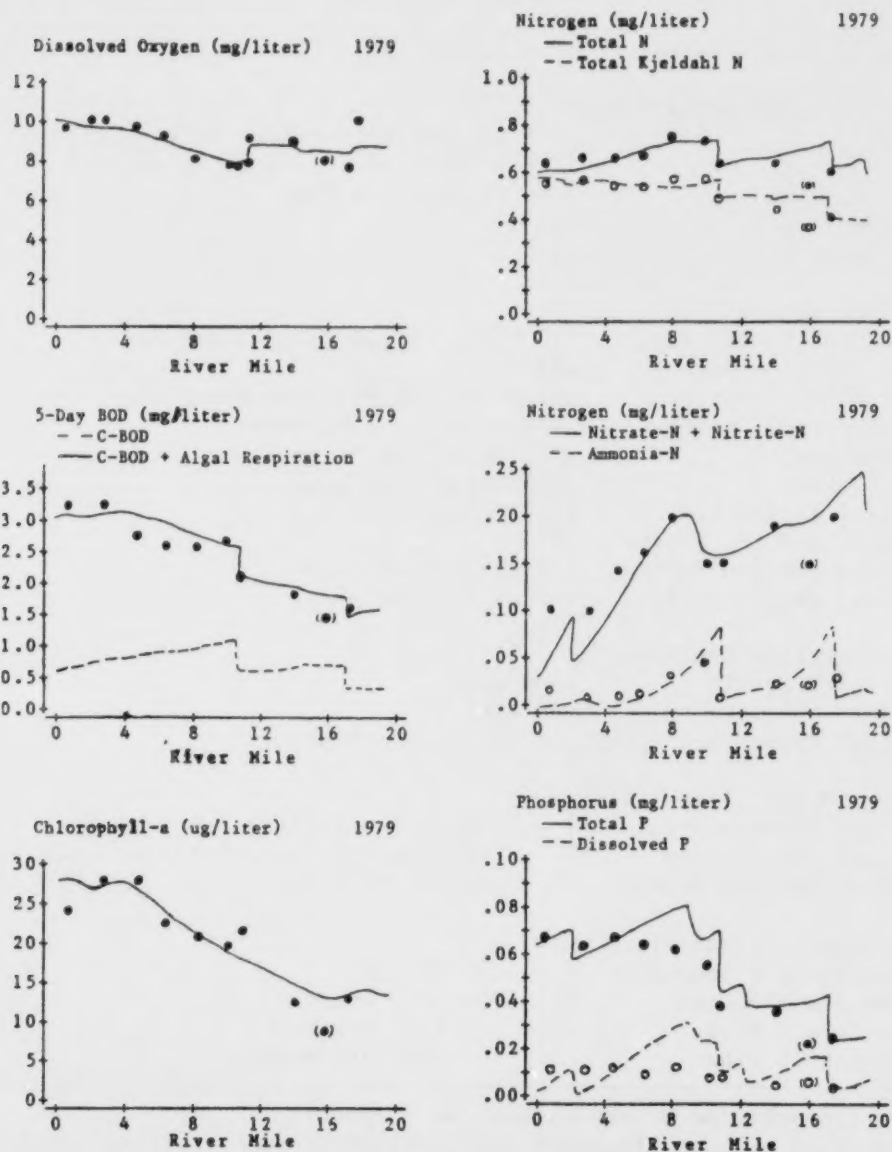
parameters are zero in other applications if some initials are given
parameters are used in all applications if no initials are given
ranges refer to reach-specific values, all rates at 20 degrees C

* plus wetland benthic loading component

** benthic sources of phosphorus and ammonia in impounded reaches

Figure 2

Observed and Predicted Water Quality Profiles
Lower Winooski River 1979



temporal sampling frequencies for all water quality variables. In this application, reasonable simulation of observed chlorophyll and afternoon oxygen (between predicted daily mean and daily maximum) profiles was achieved with minor adjustments in the parameters used in the Winooski simulations.

The model has been recently applied to the Sudbury/Concord Rivers in Massachusetts (Walker, 1983), as part of an assessment of the potential environmental constraints involved in diverting waters from the upper watershed for water supply purposes. The study extends over 51 river kilometers, 44 of which consist of a shallow impoundment surrounded by wetlands, including the Great Meadows National Wildlife Refuge. Both the model and monitoring data indicate that water quality conditions are controlled largely by the hydraulic geometry and by interactions with tributary wetlands and that point sources are of minor importance. Hydraulic simulations using HEC-II and detailed channel cross-section measurements at 278 locations have been used to define hydraulic geometries in water quality model.

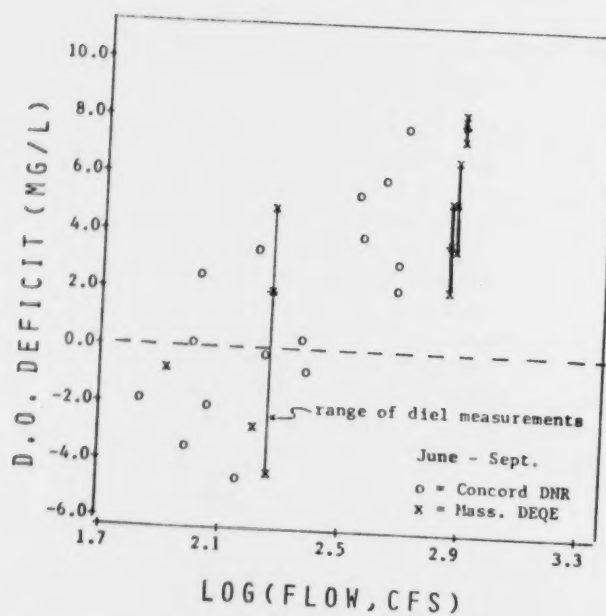
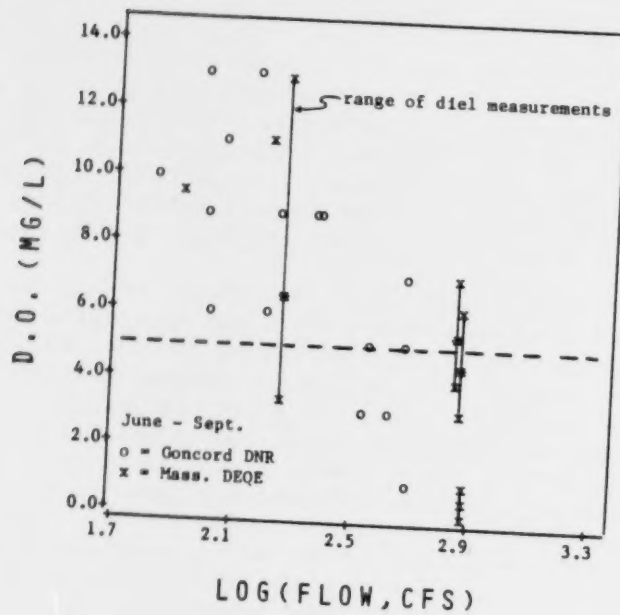
As a result of the channel and floodplain morphometry and backwater effects, the river undergoes relatively large changes in width as a function of flow in certain reaches. Bottom sediments in the channel and overbank areas are highly organic in nature and reflect the export, settling, and decay of organic materials from adjacent wetlands. Figure 3 depicts variations in dissolved oxygen and dissolved oxygen deficit as a function of river flow measured during summer months at the lower end of the impounded area most strongly influenced by adjacent wetlands. A unique aspect of this system is that dissolved oxygen levels tend to be lower (and deficits, higher) during high-flow periods. The relationships in Figure 3 reflect the combined influences of (1) higher organic loadings during high-flow periods; (2) higher benthic demands in overbank areas; (3) less algal growth because of the increased flushing rate during high-flow periods; (4) die-off and decay of wetland vegetation during infrequent summer flooding events. The last condition results in the most severe water quality conditions, including depression of oxygen levels below 2 mg/liter over most the impoundment. A significant fishkill was reported in 1938 following the largest recorded July flood.

Preliminary calibration of the net benthic oxygen demands in each model reach under high-flow and low-flow conditions indicated that demands increased with the ratio of tributary wetland area to river surface area and increased with average basin runoff. Accordingly, a simple mass balance model linking the apparent benthic demand to tributary wetland areas and runoff rate has been used to represent wetland interactions. The calibrated wetland organic export rate of .17 grams of oxygen demand per square meter per day represents only about 1.3 to 2.1 percent of the literature range for net primary productivity of freshwater emergent macrophytes on fertile sites in temperate regions (Wetzel, 1975). The remainder of the organic matter produced in the wetlands is apparently decomposed in place or flushed downstream with little decomposition. Another wetland interaction considered in the model is benthic uptake of nitrate (.1 g/m²-day) in overbank areas and is attributed to plant uptake and denitrification supported by organic substrates, as observed in other wetland systems (Kadlec and Kadlec, 1978).

The model has been calibrated and tested against data from three intensive surveys (1) August 1973, summer low-flow; (2) July 1973, summer

Figure 3

Summer Dissolved Oxygen Concentration and Deficit
at Route 117 vs. Concord River Flow at Lowell



flood; and (3) June 1979, late-spring flood. While the average flows were similar in the July 1973 and June 1979 surveys, the major hydrologic difference is that the former occurred a week after a summer storm event of about seven-year frequency, whereas flows were decreasing seasonally during the latter, when only flood-tolerant vegetation would have developed in adjacent wetlands. Over most of the impoundment, diel oxygen fluctuations were less than 1 g/m³ during the July 1973 survey, as compared with a range of 2 - 8 g/m³ for the other surveys; this reflects a suppression of plant photosynthesis associated with summer flooding.

Observed and predicted daily mean oxygen profiles for each survey are shown in Figure 4. The severe conditions during July of 1973 have been simulated by setting benthic photosynthesis rates to zero, as compared with a 1.5 - 10 g/m²-day range calibrated to various reaches for the other survey periods. All other model parameters are fixed for the three simulations. An oxygen sag attributed to high benthic demands and low velocities is apparent below river kilometer 48, the approximate upper end of the backwater effects created by the impoundment at river kilometer 7.

Observed and predicted chlorophyll-a profiles are shown in Figure 5. Using the same kinetic and stoichiometric parameters used in the Lower Winooski algal simulations, the model simulates the peak algal densities observed during the low-flow and spring-flood surveys with reasonable accuracy, especially considering that the observed points are based upon single grab samples. For the low-flow survey, the chlorophyll profile is over-predicted below the Assabet River (RKM 25). The time-of-travel through the impoundment during that survey was about 20 days and it is possible that the lower observed chlorophyll levels reflect higher flow conditions previous to the survey. Other possible explanations include effects of zooplankton predation (more likely to be important at long residence times but not simulated by the model), or shading by floating duckweed which have been observed in this portion of the river. Despite this problem, the model adequately simulates the observed chlorophyll-a profiles in the upper end of the impoundment, where the oxygen sag is located.

Conclusions

This paper has described adaptations and applications of QUAL-II to three New England river basins. Incorporation of organic nutrient compartments and updating of algal growth kinetics increase the realism and generality of the model. Interfacing the model output with SAS and plotting routines facilitates calibration, testing, and statistical analysis of observed and predicted water quality profiles. The revised code can be used on microcomputers or mainframes with FORTRAN capability.

Reasonable generality is indicated by model calibration to observed water quality profiles with minimal adjustment in key parameter estimates from one application to another, especially in those parameters which determine nutrient and algal profiles. Relatively large variations in some parameters, including nitrification rate, benthic oxygen demand, and benthic photosynthesis, reflect inherent limitations in this type of model and dictate needs for calibration and testing in each application. Modifications of the code to permit systematic sensitivity analysis (Walker, 1982a) would improve user perspectives of key processes and assumptions in a given model application.

Figure 4

Observed and Predicted Mean Dissolved Oxygen Concentrations
Sudbury/Concord Rivers

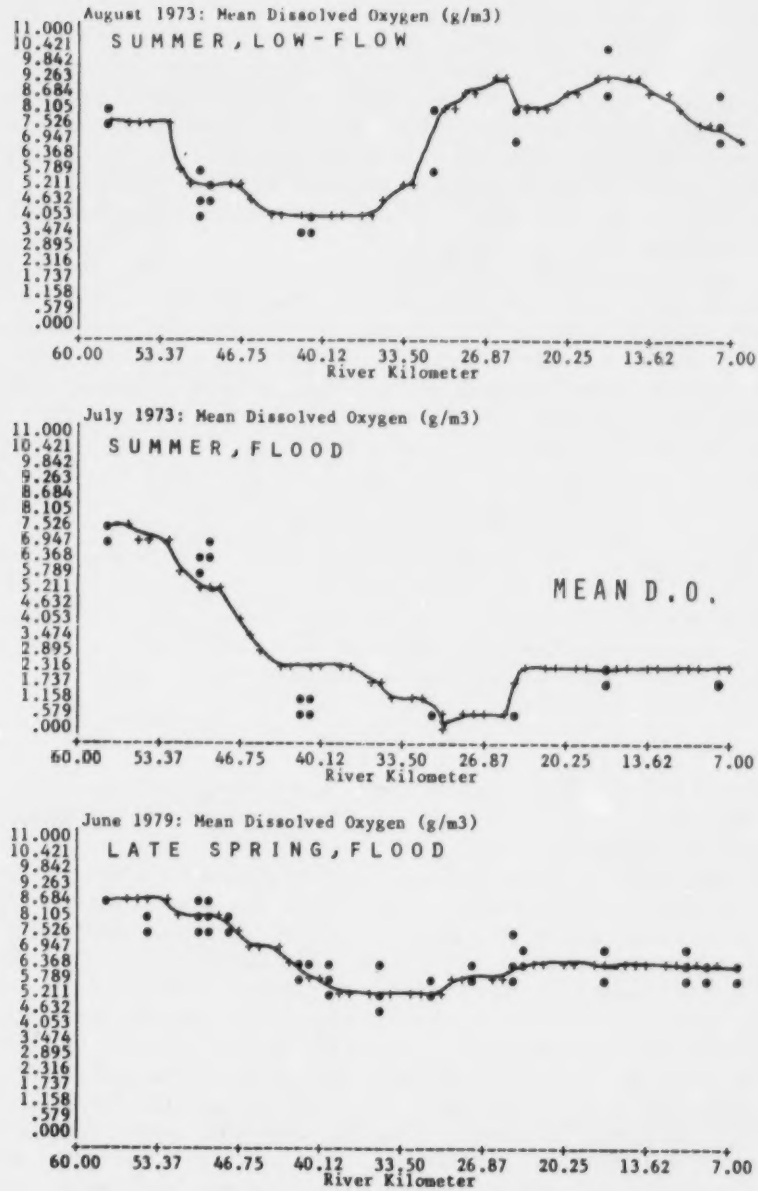
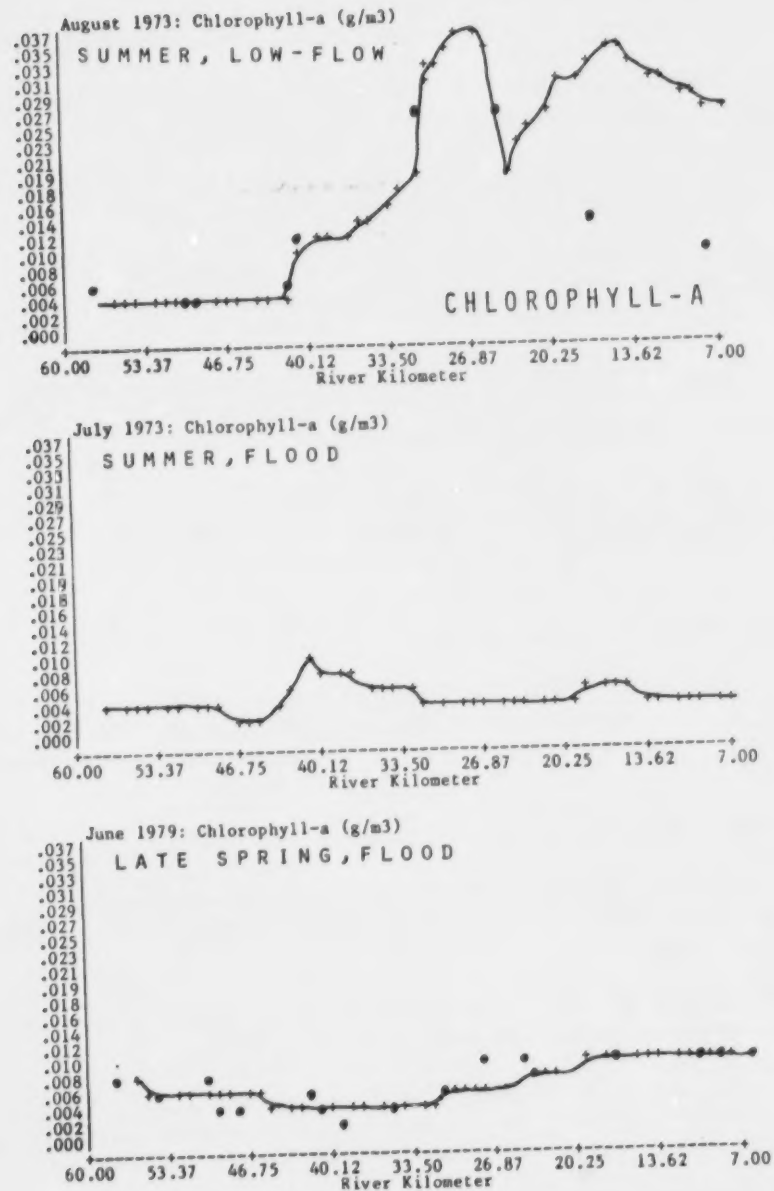


Figure 5

Observed and Predicted Chlorophyll-a Concentrations
Sudbury/Concord Rivers



References

DiTorro, D.M., "Algae and Dissolved Oxygen", in "Basic Models of Natural Systems", prepared for U.S. Environmental Protection Agency by Manhattan College, New York, 1968.

DiTorro, D.M., Thomann, R.V., O'Connor, D.J., and Mancini, J.L., "Estuarine Phytoplankton Biomass Models - Verification Analyses and Preliminary Applications", in The Sea, John Wiley and Sons, Inc., New York, 1977.

Elder, J.W., "The Dispersion of Marked Fluid in Turbulent Shear Flow", Journal of Fluid Mechanics, Vol. 5, pp. 544-560, 1959.

Fischer, H.B., E.J. List, R.C.Y. Koh, J. Imberger, and N.H. Brooks, Mixing in Inland and Coastal Waters, Academic Press, New York, 1979.

Kadlec, R.H. and J.A. Kadlec, "Wetlands and Water Quality", in Wetland Functions and Values: The State of Our Understanding, American Water Resources Association, November 1978.

Meta Systems, Inc., "Documentation for the Meta Systems Version of the QUAL-II Water Quality Simulation Model", prepared for U.S. Environmental Protection Agency, Water Planning Division, 1979.

National Council for Air and Stream Improvement, Inc., "A Review of the Mathematical Water Quality Model QUAL-II and Guidance for Its Use", Stream Improvement Technical Bulletin No. 335, 1980.

O'Connor, D.J. and W.E. Dobbins, "Mechanism of Reaeration in Natural Streams", Trans. ASCE, Vol. 123, 1958.

Roesner, L.A., et al., "Computer Program Documentation for Stream Quality Model (QUAL-II)", U.S. Environmental Protection Agency, Center for Water Quality Modeling, Athens Environmental Research Laboratory, Georgia, EPA-600/9-81-014, 1981.

SAS Institute Inc., "SAS Users Guide", Raleigh, NC, 1979.

Streeter, H.W. and E.B. Phelps, "A Study of the Pollution and Natural Purification of the Ohio River", U.S. Public Health Service Bulletin 146 (reprinted 1958), 1925.

Texas Water Development Board, "Simulation of Water Quality in Streams and Canals", Program Documentation and User's Manual, 1970.

VanBenschoten, J.B. and W. W. Walker, "Calibration and Application of QUAL-II to the Lower Winooski River", draft manuscript submitted to Water Resources Bulletin, American Water Resources Association, August 1982.

Vermont Agency of Environmental Conservation, "Lower Winooski River Wasteload Allocation Study - Part A: Report of Data", Department of Water Resources and Environmental Engineering, Montpelier, December 1980.

Vermont Agency of Environmental Conservation, "Lower Winooski River Wasteload Allocation Study - Part B: Mathematical Modeling Report", Department of Water Resources and Environmental Engineering, Montpelier, January 1982.

Walker, W. W., "A SAS Interface for QUAL-II", prepared for U.S. Environmental Protection Agency and Vermont Agency of Environmental Conservation, 1980.

Walker, W. W., "Calibration and Application of QUAL-II to the Upper East Branch of the Sebasticook River between Corinna and Coburn", prepared for Kleinschmidt and Dutting, Inc., Pittsfield, Maine, 1982.

Walker, W.W., "A Sensitivity and Error Analysis Procedure for Lake Eutrophication Modeling", Water Resources Bulletin, American Water Resources Association, Vol. 18, No. 1, pp. 53-61, February 1982a.

Walker, W. W., "Downstream Water Quality Impacts of Diversions from Sudbury Reservoir - Model Calibration and Testing", prepared for Interdisciplinary Environmental Planning, Inc., Parsons Brinkerhoff Quade & Douglas, Inc., and Metropolitan District Commission, Commonwealth of Massachusetts, January 1983.

Water Resources Engineers, Inc., "Progress Report on Contract No. 68-01-0713, Upper Mississippi River Basin Model Project", prepared for U.S. Environmental Protection Agency, September 1972.

Wetzel, R.G., Limnology, W.B. Saunders Company, Philadelphia, 1975.

Zison, S.W., W.B. Mills, D. Deimer, C.W. Chen, "Rates, Constants, and Kinetics in Surface Water Quality Modeling", U.S. Environmental Protection Agency, Athens Environmental Research Laboratory, Georgia, EPA-600/3-78-105, December 1978.

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

A REVIEW OF MODEL USE IN EVALUATING
NONPOINT SOURCE LOADS FROM FOREST MANAGEMENT ACTIVITIES

George G. Ice¹ and Raymond C. Whittemore²

I INTRODUCTION

The water quality of runoff from forest land is a topic of continuing interest; particularly regarding the influence of management activities. Models and predictive techniques are now being used to compare alternative management strategies. This paper will discuss: (a) the unique regulatory requirements that mandate predictions of nonpoint source (NPS) contributions for forest lands; (b) procedures which are being developed or used to predict sediment loading to streams; (c) areas of continuing need for modelling development; and (d) forest industry research to improve our understanding of forest-water quality relationships and modelling.

II NEED FOR MODELLING YIELDS FROM SILVICULTURAL NONPOINT SOURCES

There is currently a urgent need to validate procedures to predict water quality, streambed and stream biota responses to forest practices. In part this need results from the Clean Water Act and the diverse silvicultural nonpoint source (NPS) control programs developed under Section 208. However even more pressing are the recent questions that have been raised about modelling in regard to state forest practice acts, state water quality standards, and legislation covering planning on National Forests. In addition there is a continuing evolution in the perspective of forest planners regarding NPS loading which is further complicating both modelling approaches and solutions.

Like other NPS activities, the Clean Water Act required ~~that, where necessary,~~ control programs for silvicultural activities be developed by the states. These programs have emphasized the use of Best Management Practices (BMP's). The development of BMP's has required an understanding of the mechanisms influencing NPS loading. For example, in Florida recommended practices under the voluntary silvicultural NPS control program are based on a site-sensitivity classification which is a function of steepness-of-slope, soil erodibility (K-factor from the Universal Soil Loss Equation), and proximity to open water (1).

1 Research Forester, NCASI, Oregon State University, Corvallis, Oregon

2 Research Engineer, NCASI, Tufts University, Medford, MA

A recent survey of state silvicultural NPS control programs found that, in general, these programs have been well implemented and are effective in reducing NPS loading (2). However, as part of an ongoing process, changes in BMP's continue to be considered. Using Florida again as an example, some changes in guidelines have been proposed based on an understanding of "source area differences" between physiographic provinces (3). In New Mexico, an extensive BMP development program is ongoing. Robert Brozka, Water Quality Project Forester for the New Mexico Forestry Division writes that,

"... soil movement was quantified by means of the Modified Universal Soil Loss Equation using procedures described in the EPA handbook titled 'An Approach to Water Resource Evaluation of Non-Point Silvicultural Sources'. Soil movement on roads was quantified separately utilizing a mathematical sediment yield model developed specifically for forest roads. This same model is currently being adapted specifically to the State by the New Mexico Water Resources Research Institute. Data is being collected from around the State on runoff and sediment production rates with a rainfall simulator. Ultimately, different road designs and erosion control practices will be evaluated with the model so that the best alternative can be chosen." (4)

In some states, predictive procedures are being used or are proposed to be used to indicate appropriate forest management practices and compare alternatives on a site-specific basis. In Northern California, where the silvicultural NPS control program is enforced through the State Forest Practice Act, an Erosion Hazard Rating (EHR) has been used to identify areas of high erosion potential. In the past the EHR has combined both surface and mass wasting (landslide) erosion hazard. Two recent studies (one by California Division of Forestry and the other by Humboldt State University) found that the EHR was not a good indicator of observed erosion rates (5,6). In response to this, the California Board of Forest has recently adopted a new EHR for surface erosion and is in the process of developing a new EHR for mass wasting. See Table 1 for factors considered by the California surface soil EHR. Rules have recently been adopted in California which require EHRs down to 10 acre parcels in some cases.

In Idaho, USDA Forest Service operations in one watershed were recently halted because a USDA Forest Service model projected that harvesting and roading would result in sediment loading which would cause a 20 percent reduction in fishery potential. This projected 20 percent reduction in fishery potential was interpreted by the Idaho Department of Health and Welfare as violating state water quality standards that protect beneficial use.

In Oregon, a petition has been submitted to the Board of Forestry which requests that an erosion model developed by the Siuslaw National Forest for forest-wide planning be adopted as part of the Oregon Forest Practice Act to be used in evaluating Forest Operations along the Oregon Coast. The petition requests that operations be limited to those that, according to the Siuslaw model, restrict sediment loading increases to less than 200 percent of the natural rate. A further discussion of the model is presented later.

TABLE 1 SURFACE SOIL EROSION HAZARD RATING
FORM FOR STATE OF CALIFORNIA

STATE OF CALIFORNIA
DEPARTMENT OF FORESTRY
ESTIMATED SURFACE SOIL
EROSION HAZARD
FORM 1 (12/81)

I. SOIL FACTORS

A. SOIL TEXTURE	FINE	MEDIUM	COARSE	FACTOR RATING
1. DETACHABILITY	LOW	MODERATE	HIGH	
RATING	1-3	10-18	19-30	
2. PERMEABILITY	SLOW	MODERATE	RAPID	
RATING	5-4	3-2	1	

B. DEPTH TO RESTRICTIVE LAYER OR BEDROCK

	SHALLOW	MODERATE	DEEP	
	< 20"	20"-39"	> 39"	
RATING	15-9	8-4	3-1	

C. PERCENT SURFACE COARSE FRAGMENTS GREATER THAN 2MM IN SIZE INCLUDING ROCKS OR STONES

	LOW	MODERATE	HIGH		FACTOR RATING
	< 40%	40-70%	70-100%		
RATING	10-6	5-3	2-1		

SUB TOTAL ↓

II. SLOPE FACTOR

	5-15%	16-30%	31-40%	41-50%	51-70%	> 70%
RATING	1-3	4-6	7-10	11-15	16-25	26-35

III. PROTECTIVE VEGETATIVE COVER REMAINING AFTER DISTURBANCE

	LOW	MODERATE	HIGH
	0-40%	41-80%	81-100%
RATING	15-8	7-4	3-1

IV. TWO-YEAR, ONE HOUR RAINFALL INTENSITY (Hundredths inch)

See attached map	LOW	MODERATE	HIGH	EXTREME
	< 40	40-59	60-69	> 69
RATING	1-3	4-7	8-11	12-15

TOTAL SUM OF FACTORS ↓

EROSION HAZARD RATING

< 50	50-65	66-75	> 75
LOW	MODERATE	HIGH	EXTREME

THE DETERMINATION IS ↓

The Forest Service has been the developer and user of numerous sediment loading models recently because the National Forests are subject to special planning requirements under the National Environmental Policy Act of 1969 and National Forest Management Act (NFMA). Under NFMA the USDA Forest Service is required to make projections about resource outputs resulting from management options. The results of these decisions will influence the availability of timber, particularly for the Western United States. However a recent evaluations about available predictive techniques for forest resources found that "... it is impossible to accurately predict [] on-site and offsite effects ..." and the land manager is currently "... unable to define limitations to use, if and how management practices might be applied, and possible trade-offs in the various uses and values, even though he is required to do so for environmental analysis reports" (7).

On top of these regulatory and legislative requirements for the use of models in forest management planning, the forestry community (particularly in the West) has recently begun to consider the consequences of multiple forest operations on water quality. A California Board of Forestry task force has raised the concern that "... effects may appear to be insignificant when a harvesting project is viewed alone, but may become cumulative and cause demonstrable damage when the effects of other harvesting projects are considered" (8). Some of the potential cumulative effects that have been propose include: (a) peak flow increases from snow accumulation and melt rate changes in cleared areas, (b) peak flow increases due to soil compaction and altered drainage networks, (c) channel changes due to accelerated inputs of large sediment and organic debris, and (d) channel changes due to removal of large wood debris and streamside vegetation (9).

In addition to the consideration of multiple operations, forest planners are now being required to project not only changes in sediment concentration (the traditional NPS measure for forest practice impacts on water quality) but also changes in channel conditions and ultimately the beneficial uses of the water.

From these examples we can summarize that there continues to be increasing sophistication demanded in projecting the water quality consequences of forest activities at both the state and federal level. Models are being used both in long-term planning on federal lands and by states as a tool to identify general management practices which are appropriate for use as BMP's. In order for planning decisions to be the best possible it is necessary that the models being used be tested and their limitation considered.

III PROCEDURES TO EVALUATE WATER QUALITY RESPONSE TO SILVICULTURAL OPERATIONS

Because sediment is considered to be the most important NPS contribution from forest activities we will concentrate our discussion on this parameter. It is recognized, however, that in other more specific situations nutrients, biochemical oxygen demand, temperature, and herbicides can result in localized non-point source pollution problems.

A. WRENSS

In 1980, EPA published "An Approach to Water Resources Evaluation of Nonpoint Silvicultural Sources" (WRENSS) (10). WRENSS was written by forest hydrologists with the USDA Forest Service and is now being used (in various modified forms) by several states and National Forests Management groups. Because WRENSS involves many of the modelling elements currently used we will provide a brief description of the sediment prediction components.

- (1) Overview - WRENSS models changes resulting from forest activities that cause increases in water available for runoff, and also increases in both surface erosion and landslides. All these modifications are then considered using sediment-rating curves to evaluate potential channel geometry changes.
- (2) Water Quantity - Water available for runoff is modeled using a water-balance approach where the change in available water is a function of changes in interception and evapotranspiration (ET) resulting from vegetation removal. Modifications in interception and ET are a function of the change in leaf area index (LAI) or forest cover density (CD). Response of discharge is given in terms of either a seasonal hydrograph for snow dominated regions or a flow duration curve for rain dominated regions.
- (3) Surface Erosion - Sediment loading to streams is a function of three processes; surface erosion, sediment from landslides (mass wasting), and channel scour. Surface erosion is calculated with a version of the Universal Soil Loss Equation which has been modified for application to forest sites.

The modified soil loss model (MSLE) is:

$$A = R K L S V M$$

where:

- A = The estimated average annual soil loss per unit area in tons/acre.
- R = The rainfall factor, usually expressed in units of rainfall-erosivity index, EI, and evaluated from an iso erodent map.
- K = The soil-erodibility factor, is usually expressed in tons/acre/EI units for a specific soil in cultivated, continuous fallow, tilled up and down the slope.
- L = The slope length factor is the ratio of soil loss from the actual field slope length to that from a 72.6-foot (22.1 m) length plot.
- S = The slope gradient factor, is the ratio of soil loss from a given field gradient to that from a 9-percent slope.
- VM = The vegetation-management factor, is the ratio of soil loss from land managed under specified conditions to that from the fallow condition (10).

A delivery ratio is calculated from the site conditions to predict the amount of soil loss (A) actually reaching the stream system. This portion of WRENSS might be used to predict surface erosion from site-preparation activities as are shown in Figure 1.



FIGURE 1
BEDDING AS A SITE
PREPARATION TECHNIQUE
IN FLORIDA

(4) Landslides - In the West, landslides can be major contributor to the sediment loading of forest streams. See Figure 2. Of the two general types of landslides (slump-earth flows which are deep, slow movements in plastic soils, and debris avalanche-debris flows which are rapid, shallow failures) it is the debris avalanche-debris flow type which is most influenced by forest activities.

In order to predict landslide rates using WRENSS, a similar nearby site which has previously experienced comparable management activities must be inventoried to determine landslides under undisturbed and man-induced conditions. Site and practice hazard similarity can be judged using a hazard index based on site conditions. From this first inventory, an accelerator factor (F) is developed where:

$$F = \frac{\text{man-induced failures}}{\text{natural failures}}$$

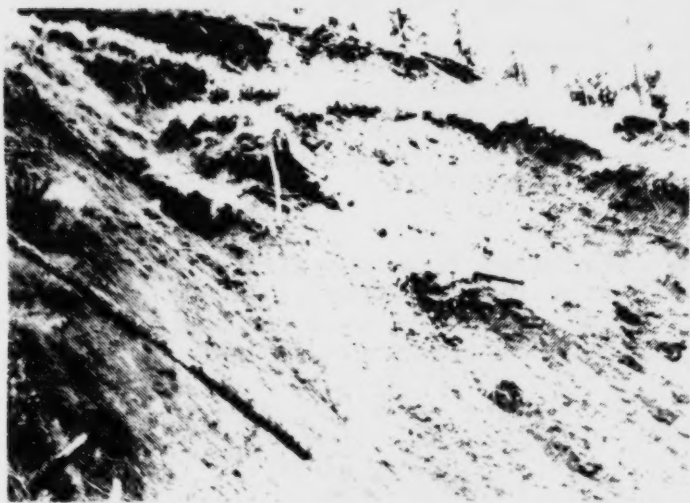


FIGURE 2
MASS WASTING IN THE
OREGON CASCADES

Another inventory of the site being modeled, provides the natural delivery rate which is multiplied by the accelerator factor to yield the delivery of man-induced mass failure material.

(5) Total Potential Sediment and Channel Modification - System modifications are combined to predict changes in total sediment. The following steps are involved.

- (a) From the pre and post management flow-duration curves and a sediment rating curve developed by stream measurements, pre and post management suspended sediment loads are developed. [Stream stability may be used in post-management sediment rating curve adjustments]. This step accounts for channel-derived sediment load increases. See Figure 3.
- (b) Bedload sediment rating curves are developed as above and the same type of analysis is used to generate pre and post management bedload rates.
- (c) Coarse and fine sediment contributions are separated from material delivered to the stream from mass wasting.

(d) Total suspended sediment load is determined as the cumulative yield from rating curve increases, surface erosion increases, and the wash load from mass failures. All surface erosion material is assumed to be suspendable.

(e) Total sediment available for transport is determined by including bedload and coarse mass-wasted material.

(f) The potential for channel modification are assessed by comparing maximum bedload transport to introduced coarse material.

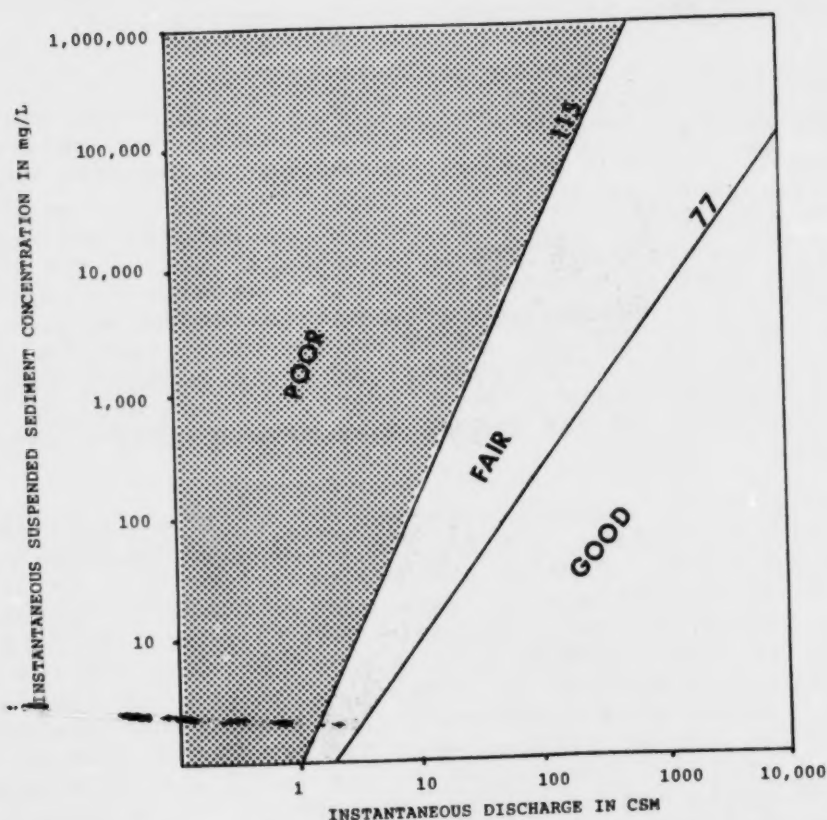


FIGURE 3

SEDIMENT RATING CURVE WITH STREAM STABILITY
RATING (SSR) ADJUSTMENTS - DEVELOPED FOR FOX
PLANNING UNIT, SIX RIVERS NATIONAL FOREST, CALIFORNIA
(POOR SSR 115; FAIR SSR 115 AND 75; GOOD SSR 75)

B. NCASI Activity Involving WRENSS

Because WRENSS is being applied to an increasing number of assessments, this procedure and its derivatives will be the subject of numerous performance evaluations.

Recently, a portion of WRENSS was used to predict baseline sediment yields from undisturbed basins. As previously mentioned, the NFMA has necessitated that sediment yields along with other factors, be considered for planning purposes on National Forests. For this reason methods which predict changes in sediment yield are being developed and applied for specific conditions. In 1981, the USDA Forest Service Northern and Intermountain Regions prepared a working draft of procedures to predict sediment yields. However, one problem was the lack of baseline data for most sites to show sediment yields prior to harvesting (12).

A method was proposed in this document which would estimate natural sediment yields. Data from undisturbed watersheds (representing low to high sediment producing landscapes) were reported to show a range of 10 to 100 tons/mi²/year. For undisturbed forest watersheds "... the source of natural sediment is primarily stream channel erosion processes" (12). The authors concluded that "since natural surface erosion is considered insignificant, the variation in natural sediment is assumed attributable to differences in mass erosion hazards and delivery differences" (12). Therefore, using the boundary values of 10 to 100 tons/ mi²/year a relationship between average natural sediment yield and the WRENSS mass erosion hazard rating was developed.

In order to rapidly evaluate this natural sediment yield estimating procedure, a USDA Forest Service Pacific Northwest Region document, "Erosion and Sedimentation Data Catalog of the Pacific Northwest" was utilized (13). The catalog summarizes the results and site characteristics for 19 small watershed studies as well as other erosion plot and large-basin studies. Several of the summarized studies include data for control (undisturbed) basins. Using site characteristics listed and knowledge of these watersheds, hazard ratings were developed and plotted against measured yields. See Figure 4. The results showed that the approach could not be applied throughout the Northwest. Further, the hazard index used could not discriminate between wide ranges of average outputs even for nearby watersheds.

Although there are many reasons why short periods of sediment monitoring might not agree with the hazard index, the need to explore the proposed relationship is apparent.

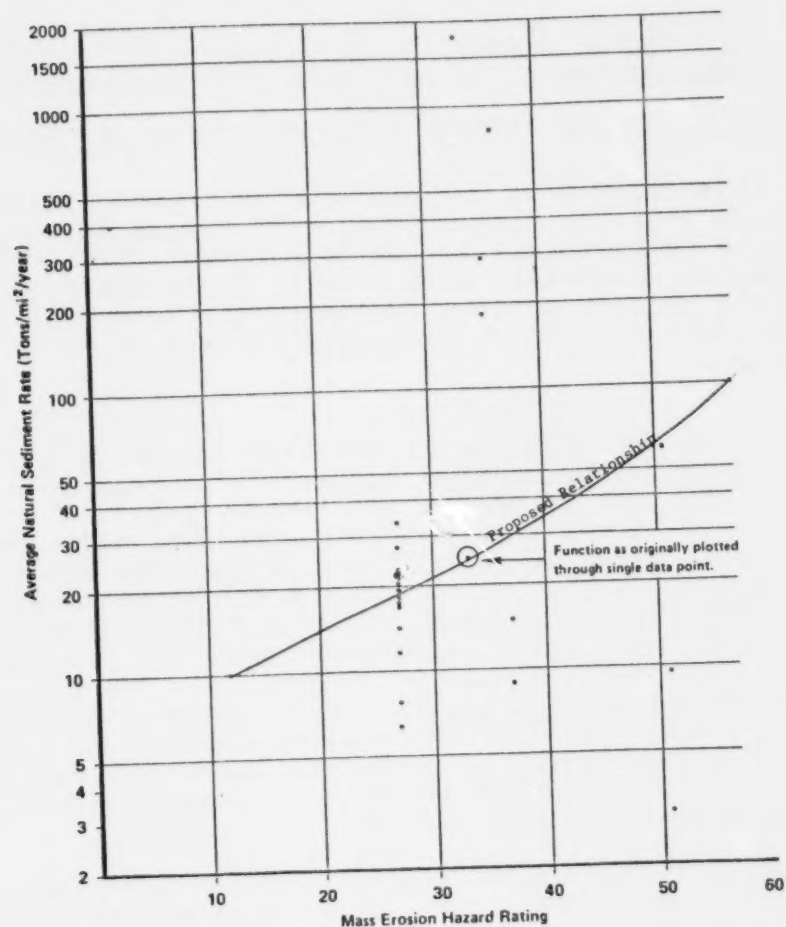


FIGURE 4
COMPARISON OF PREDICTED TO ACTUAL
SEDIMENT YIELDS FOR NORTHWEST STUDIES (20)

NCASI is now collecting information from landslide surveys which can be used to evaluate predictive methods. This information is now being organized into a catalog similar to the "Erosion and Sedimentation Data Catalog of the Pacific Northwest" which was referred to earlier. One example of the arrangement of information is shown below for a survey of landslides in the Mapleton Panger District. See Table 2.

Until recently WRENSS has involved a laborious, hand calculation procedure. The USDA Watershed Systems Development Group in Fort Collins, Colorado has just released a draft copy for review of computerized WRENSS solutions, WETT.WET and WET.SED. NCASI,

along with others, is now reviewing these programs. (Although the hydrograph portion of WRENSS is simulated by WETT.WET., the sediment portion of WRENSS is only partially reproduced.)

C. Other Procedures to Predict Sediment Yields from Silvicultural Yields

While this discussion has centered on WRENSS there are numerous other procedures being touted to assist in forest planning. A recent publication by the USDA Forest Service Pacific Northwest Region summarized more than 10 different techniques being used to quantify surface erosion, mass movements (slides), and sediment delivery (14). See Table 3 for a short description of these models.

The Siuslaw National Forest provides another example of the number of models being proposed for use in forest planning. Passing over the erosion prediction options provided in Table 3, hydrologist, soil scientists and fisheries biologists on the Siuslaw National Forest have developed 5 separate procedures to predict sediment loading to streams and the response of anadromous fish as a result of forest roading and harvesting. These procedures included: (a) a forest-wide planning procedure; (b) a district-wide procedure to estimate cumulative effects; (c) a district procedure which estimates landslide-only sediment loading; (d) a site-specific procedure which requires on-site estimates of the effectiveness of mitigative measures, and (e) a stream-threshold procedure to estimate when unacceptable sediment loading occurs to the stream system.

As previously discussed, the forest-wide procedure for the Siuslaw National Forest has been proposed as a predictor for defining appropriate practices under the Oregon Forest Practice Act. The Siuslaw Model benefits greatly from numerous on-site studies and landslide inventories and this empirical model is based in part on these relationships. Components predicted include natural erosion rates, in-unit failures and volumes, road related failures and volumes, surface erosion rate (dry ravel from broadcast burning) and delivery, and effectiveness of protective measures (leave areas). The procedures weaknesses include lack of validation, lack of discrimination between a wide range of practices, and the need for gross generalizations about site considerations.

NCASI has developed computer programs to solve the Siuslaw National Forest procedures as well as many of those presented by the Pacific Northwest Region in order to compare predicted sediment loading rates with existing data sets.

TABLE 2 EXAMPLE OF MASS WASTING CATALOG

Inventory of Mass Erosion in the Mapleton Ranger District
Siuslaw National Forest and Pacific Northwest Forest and Range Experiment Station

Location: Selected units in the Mapleton Ranger District, Siuslaw National Forest, Oregon were evaluated. See Mapleton District - Ketcheson and Froehlich for the location of the of the Mapleton District.

Objective: With improved road building, in-unit slides are suspected as possibly becoming more important (proportional) to the overall sediment budget for steep terrain sites. This is especially true because road right-of-ways occupy only a fraction (approx. 8%) of the overall area being harvested. Therefore, an inventory of slides was made to determine the number of failures in forested, in-unit (clearcut) and road right-of-way sites. The focus of the study was on the most slide-prone soil types in the Mapleton District.

Site Characteristics: See Mapleton Ranger District - Ketcheson and Froehlich for a complete description of soil types, topography, and precipitation. This inventory focused on SRI soil type 47 which is the most slide prone in the Mapleton District (hazard rating of 5) and has shallow, coarse-textured soils with steep, deeply-incised slopes.

Procedures: Color air photos (scale = 1:15,840) made in 1972 were used to determine road and in-unit slide frequencies. Field measurements were used to establish the size distribution of these slides along with dating by dendrochronological methods. In-forest erosion rates were determined exclusively using field observations. Slide volumes less than 10 yds³ were excluded. The inventory covered time periods of 10, 15, and 20 years, respectively, for clearcut (in-unit), road right-of-way, and natural failures. No accounting was made of debris torrents following slide entrance into the stream channel.

Results: Comparison of in-field and air photo methods showed good agreement for clearcut, and road-in-clearcut sites, but poor agreement for road-in-forest sites (due to shadows) so in-field techniques were used for the forest and road-in-forest calculations. Clearcuts adjacent to roads did not significantly increase the slide rate of road right-of-ways. Photo inventory methods were found to detect 40 yds³ or larger slides, while in-field measurements detected 10 yds³ or larger failures (1/4 of in-unit slides 40 yds³ or less). The frequency of events for in-forest sites was found to be 20x the rate for other sites studied in the Northwest, but because the average volume was 4x the volume found in other studies, the in-forest slide erosion rate was similar. A decrease in importance of roads between this study and the initial Mapleton landslide inventory was noted (change in the ratio of clearcut area to road failures from 7.3 to 31).

In an Appendix report, debris torrents were surveyed for Cedar Creek which is a tributary to Sweet Creek and the Siuslaw River, 5.5 miles SW of Mapleton. The analysis used cumulative acre-years in order to account for changing land-uses of the period of records. The authors concluded that a similar adjustment to the Mapleton data could increase the man-caused rate of sliding 2 to 4 times.

LANDSLIDES FOR FORESTED, CLEARCUT, AND ROADED SITES

	Units Associated	Road Associated	Natural Forested
<u>% Slope Class</u>			
Percent of Failure			
0-58	3	3	15
59-70	18	55	33
71-84		55	38
84-100	25	19	8
100-119	4	3	2
119+			
<u>ASPECT</u>			
Percent of Failure			
N	11	13.8	8.2
NE	10	20.7	4.1
E	14	12.6	14.3
SE	13	5.7	12.2
S	15	12.6	16.3
SW	12	9.2	14.3
W	18	16.1	12.2
Other	9	9.2	18.4
<u>SRI Classification</u>			
Volume of Failure yds ³ /acre/year			
47	0.60	18.5	0.15
others	0.20	5.1	0.28
average	0.33	8.4	0.17

DEBRIS TORRENTS ON CEDAR CREEK DRAINAGE

	Cumulative Acre-Years	Debris Torrents	Frequency (Events/Acre-Yr)	Frequency to Debris Torrents for Forest Areas
Forest	34,344	5	0.00014	1
Clearcut	10,751	2	0.00074	5.3X
Road right-of-way	1,415	8	0.0056	40.X
<u>Reference:</u> Swanson, F.J., and Swanson, W.M., "Inventory of Mass Erosion in Mapleton Ranger District, Siuslaw National Forest," USDA For. Serv. Siuslaw N.F. and Pacific Northwest For. and Ranger Exp. Stat. (September 1977).				

TABLE 3 AVAILABLE MODELS AND TECHNIQUES FOR COMPUTING ON-SITE
EROSION, MASS MOVEMENT, SEDIMENT DELIVERY INDICES,
AND EROSION HAZARD RATINGS AS LISTED BY THE
USDA FOREST SERVICE PACIFIC NORTHWEST REGION (14)

<u>Technical Name</u>	<u>Description</u>	<u>Computer Solutions Available</u>
EROSON	Program computes maximum erosion hazard rating in inches of soil lost per year based on slope, precipitation, and erodibility coefficients, and a basic erosion rate. Cover conditions are then used to estimate actual soil loss.	Y
ONEROS 3	Modified Musgrave method which is adapted to forest applications and used to predict sheet-erosion in tons/acre. Cully erosion and delivery of on-site erosion to streams can be included in calculation.	Y
USLE	Calculates sheet erosion based on rainfall, soil erodibility, slope-lengths, cover and erosion control factors in tons/acre (or desired units).	Y
MSLE	Modified USLE which uses precipitation, soil erodibility, topographic, and vegetation management factors to predict on-site erosion in tons/acre. (Modified for forest use).	N
Megahan Time Trend Erosion Model	Forest level planning model which distributes erosion over time in tons/acre based on long-term normal erosion (no new disturbance), material available to erode after disturbance, and potential recovery rate.	Y
Region 5 Erosion Hazard Rating System	Point total based on site characteristics defines hazard rating.	N
Region 1 and 5 Guidelines for Predicting Sediment Yields	Erosion predicted as a function of land type and management activity. Guidelines include functions to deliver eroded material to streams and route it to critical areas.	N
WRENSS Mass Erosion Hazard Rating	Two erosion hazard ratings (for debris avalanche and slump-earthflow types failures) based on site conditions and a point scale for component hazard	N
WRENSS Sediment Delivery Model	Sediment delivery index based on available water, texture of eroded material, ground cover, slope gradient, slope shape, surface roughness, and delivery distance.	Y
Region 5 Sediment Delivery Model	Delivery of sediment is a function of soil erosion hazard rating, drainage dissection, slope position, vegetation cover, and entrainment coefficient.	N

IV AREAS OF CONTINUING NEED FOR MODELLING DEVELOPMENT

A general understanding of many of the basic mechanisms controlling watershed-sediment response to forest-management activities has been developed and this understanding is incorporated into some of the predictive techniques discussed. For example, WRENSS has identified key components including channel scour, mass wasting, and surface erosion which can contribute to sediment load changes. However, because of the complexity of forest systems and the difficulties of quantifying those systems (particularly the hydrologic cycle), no single model has yet proven to be universally acceptable. In fact some models are being used prematurely without thorough testing simply because immediate answers and management decisions are required for planning.

The further development of models to predict NPS response to forest practices will require five steps. First our understanding of some of the mechanisms currently modelled as cause-and-effect need to be refined. For example, several models use sediment-rating curves to predict sediment load response to forest practices by predicting hydrograph changes following harvesting. Yet, it is known from sediment concentration-flow patterns for season, storm sequence, and position in the storm hydrograph, that flow is only one of the independent variables influencing sediment concentration. When discharge is increased, as a result of reduce evapotranspiration for example, the sediment-rating curve relationship could overestimate sediment loading because the rainfall-intensity factors generating material available for transport are overestimated by the flow parameters. Conversely, the sediment-rating curve relationship could underestimate sediment loading because practices could make more material available for transport. Further examples can be drawn from the bulk of research dealing with infiltration and interflow (22).

Second, key mechanisms need to be modelled which have been avoided to this point. An example is the lack of groundwater modelling for mass wasting analysis. A special soils task force reporting to the Oregon State Forester concluded that "nearly all slope failures in the forest environment occur in response to a higher than average pore water pressure condition..." and therefore the development of engineering groundwater models is necessary to improve road location and design (15).

Hewlett (21), in a recent paper on modelling philosophy, points out that hydrological modellers are currently "plagued with an embarrassment of riches" for journals contain numerous attempts to explain and/or model individual hydrological processes without providing a clear-cut choice for managers to implement. Hewlett

further suggests that decision making should be the "heart of the research enterprise". Models which simulate nature accurately may become inefficient in application to decision making because the cost is prohibitive for collecting input meteorological and/or hydrological data, and measurement of model parameters.

Difficulties may arise when users attempt to interpret cause and effect relationships from a model which was not designed to deterministically simulate the effect of interest or concern. A surge in concentration of an aerially applied pesticide in storm flow, for example, may be attributed to basin-wide flushing if an overland flow model is used to fit the hydrograph. The same effect, however, could be attributed to stream channel factors if a subsurface storm water flow model is used. Both models may fit the hydrograph for predictive purposes, but neither may imitate the actual runoff process.

Third, complete models providing the linkage between forest activity, site conditions, and the water quality factors upon which decisions are based need to be provided. In reviewing research priorities, Brown noted that the next step for research is "...integrated, process-level studies of system function. For example, instead of measuring sediment concentration at the outlet of a small watershed after clearcutting, sediment production will be defined as a function of soil disturbance, soil characteristics, terrain, and climatic variables. The transport and deposition of sediment in the stream channel will be defined as a function of the hydraulic factors involved. These conditions then will be related to the variables that influence aquatic organisms" (16). Models based on fundamental factors and directed at the key response parameters will be more transferable and useful. This system's engineering approach is clearly not a major focus of the current modelling activity being discussed.

Fourth, and perhaps most importantly, there is a need to develop workable techniques for collecting the key on-site parameters which influence site response. For example, for sites subject to mass failures, a small change in the depth of soil, presence or absence of a hardpan, groundwater drainage, or orientation of bedrock bedding planes can make the difference between a site failing or remaining intact. It needs to be acknowledged that there may be limits or uncertainty to the information we can collect and use operationally. Where site-specific information is not available for key parameters, defacto "best estimate" data is often used and the modelling problems of this approach need to be recognized and incorporated in the management decision.

Finally, models need to be thoroughly tested and documented before operational use to evaluate their accuracy and precision. This need is similar to that expressed by NCASI concerning point

source water quality models in Technical Bulletin No. 367 (23). This technical bulletin addressed the practical concerns of model selection, calibration, and verification. The findings of this study show the need to define goodness of fit criteria for evaluating models, and to develop a broad enough data base to uncover and subsequently represent mathematically major physical, chemical, and biological mechanisms in the basin before making long-range forecasts.

V RESEARCH TO IMPROVE OUR UNDERSTANDING OF FOREST WATER QUALITY MODELING

Each year since 1979, about \$9,000,000 has been spent on research to address NPS questions (17). This research includes efforts to provide the basic data necessary to model NPS mechanisms and also efforts to improve or validate models. An excellent example of industry, university, and USDA Forest Service research cooperation is centered in Gainesville, Florida under IMPAC (Intensive Management Practices Assessment Center) program. This program is filling a real information need about the effects of harvesting and site preparation in flatwoods conditions on water quality. Perhaps some of the most interesting results from this study have concerned the water quality coming from undisturbed flatwoods sites.

Many other research programs are scattered throughout the U.S. and information on these programs can be obtained in an annual NCASI Technical Bulletin. An example of the type of basic data gathering research underway is seen in the South Central U.S. Until recently there has been little or no research on controlling sediment losses from forest roads in the South Central U.S. Four road projects are now being conducted by the forest industry, states, and universities in Oklahoma, Arkansas (2), and Mississippi. See Figure 5.

Similarly, in the West, studies have been conducted to assess sediment losses from road and skid trails in Idaho, California, Oregon, and Washington. A forest industry conducted study in Idaho has demonstrated the effectiveness of scattering slash over skid trails as a BMP to minimize erosion (18). This study has also been used to compare observed sediment losses from skid trails to those predicted by commonly used sediment loss equations. See Figure 6.

A review of non-point source models applicable to the assessment of the effect of silvicultural practices on water quality is also underway at the Northeast Regional Center of NCASI. The over-land flow and catchment models described in Tables 4 through 6 are being examined for their ultimate utility to the pulp and paper industry. The information in the Tables 4 through 6 was assembled in a recent review by Ambrose (24).



FIGURE 5

FOREST INDUSTRY STUDY
OF BEST MANAGEMENT
PRACTICES FOR FOREST
ROADS IN THE SOUTH -
A PROPORTIONAL SAMPLER
COLLECTS RUNOFF FROM
THE ROAD



A



B

FIGURE 6

FOREST INDUSTRY STUDY OF EROSION FROM SKID TRAILS IN IDAHO
SHOWS THE EFFECTIVENESS OF SCATTERED DEBRIS IN REDUCING EROSION
A: SKID TRAIL WITH SCATTERED DEBRIS B: BARE SKID TRAIL

The models can be further segmented for forested areas according to sophistication by the use of a classification scheme proposed by Ambrose (24). The segmentation and classification are provided in Table 7. The models in Table 7 are the focus of current NCASI interest.

TABLE 4 PRE-SCREENED
OVERLAND MODELS-
CATCHMENT PROPERTIES

Model	Physical Properties				Spatial Properties		
	Imper- vious	Agri- cultural	Forested	Wet lands	Single	Multiple	Channel Routing
ACTMO		X				X	
AGRUM		X				X	X
Amburger et al.		X			X		
ANGWERS		X				X	X
ARM-II		X				X	
CHS		X			X		
CREAMS		X			X		
CSU		X	X			X	X
EPARRB (URI)	X	X	X	X	X		
HSF	X	X	X	X		X	X
Haith and Tubbs		X			X	X	
Hydroscience	X	X			X		
MRI	X	X	X		X		
MUMP	X	X	X	X	X		
NPS	X	X	X	X	X		
OKI-RRB	X	X	X		X		
PIR	X	X	X				
QGS	X					X	
SCRAM		X			X		
SEHSTORM	X	X	X	X		X	
SOGREAH-							
CARELAS	X					X	
Simplified							
SWMM	X					X	
STORM	X	X	X	X		X	
SWMM-Level I	X				X		
SWMM-II	X	X	X	X		X	X
Texas A&M		X				X	X
URS	X				X		
UTH-TEHM		X	X			X	
WEST							
WQAR	X	X	X	X	X		X
WRENS			X		X		X

Model	Continuous	Event	Annual	Monthly	Daily
ACTMO	X				X
AGRUM		X			X
Amburger et al.	X				X
ANGWERS		X			X
ARM-II	X				X
CHS	X			X	
CREAMS	X				X
CSU	X				X
EPARRB (URO)	X		X		
Haith and Tubbs		X			
HSF	X				X
Hydroscience	X		X		
MRI	X		X		
MUMP		X			X
NPS	X				X
OKI-RRB	X			X	
PIR	X				X
QGS	X				X
SCRAM	X			X	X
SEHSTORM					
SOGREAH-					
CARELAS		X			X
Simplified					
SWMM	X				X
STORM	X			X	
SWMM-Level I	X		X		
SWMM-II	X				X
Texas A & M	X				
URS	X		X		
UTH-TEHM	X				X
WEST					
WQAR	X		X		
WRENS	X		X		

TABLE 5 PRE-SCREENED
OVERLAND MODELS-
TIME DOMAIN*

*This reflects the effective predictive capability of the model and is considerably larger than the computational time step.

TABLE 6 PRE-SCREENED
OVERLAND MODELS-
CONSTITUENT SYSTEMS

Model	Hydro- logy	Sed- iment	Arb- itrary	Nitrogen	Phos- phorus	Carbon (organics)
ACTHO	X	X	1	X	X	
AGRUN	X	X	3			
Amberger et al.	X	X		X	X	
ANWERS	X	X	X			
ARM-II	X	X	X	X	X	X
ONS	X	X		X	X	
CREAMS	X	X	X	X	X	
CSU	X	X	X	X	X	X
EPA PRB (UR)	X	X	X	X	X	X
Health and Tubbs	X	X	X			
HSPF	X	X	X	X	X	X
Hydroscience		X	X			
MRI		X	X			
MUNP						
NPS	X	X	X			
OKI-PRB	X	X	X			
PTR	X	X	X			
QNS	X	X	X			
SCRAM	X	X	X			
SEMSTORM	X	X	10			
SDCREAM						
CARECAS	X	X	X			
Simplified SWMM	X	X	X			
STORM	X	X	X			
SWMM Level I	X	X	X			
SWMM-II	X	X	X			
Texas AGH	X	X	X	X	X	X
URS	X	X	X			
UTH-TEHM	X	X	X			
WEST	X	X	X			
WQAM			X			
WRENS		X	X			

TABLE 7 NPS MODELS AND SOPHISTICATION LEVEL

MODEL	SOPHISTICATION LEVEL
CSU	4
HSPF	4 or 3
SWMM-II	4 or 3
NPS	3
SEMSTORM	2
STORM	2
WQAM	1
WRENS	1

LEVEL 1 - Totally empirical approach to watershed hydrology and sediment generation. Useful for long term (annual) predictions. Handbook approach.

LEVEL 2 - Simplified empirical hydrology simulation via computer program. No attempt is made to account for moisture between storm events. The Rational Formula and/or SCS Curve Number approaches to runoff from impervious and pervious surfaces are used. Effective temporal resolution is 1 month.

LEVEL 3 - Watershed hydrology is calculated explicitly via a process oriented approach. Pollutant generation is handled empirically by link to sediments or the use of washoff functions. Effective temporal resolution is of the order of one day or less.

LEVEL 4 - These models treat important hydrologic, nutrient, and sediment processes in detail. Nutrient interactions can occur in both particulate and dissolved form. Detail structure is built into the model to account for various management strategies. The time resolution is of the order of one day or less.

VI LITERATURE REFERENCES

- (1) "Summary of the Current Status of Silvicultural 208 Programs-1980," Special Report No. 80-12 (December 1980).
- (2) "A Summary of Silvicultural Nonpoint Source Control Programs for the United States-1982," NCASI Special Report 83-01 (January 1983)
- (3) Olszewski, R.J., "Streamside Management Zones in Florida and the Southeast," Presented in Technical Bulletin No. 389 (November 1982).
- (4) Brozka, R.J., Water Quality Project Forester, Forestry Division, Natural Resources Department, Santa Fe, New Mexico, Personal Communication (August 4, 1982).
- (5) Hauge, C.J., Furniss, M.J. and Euphrat, F.D., "Soil Erosion in California's Coastal Forest District," Calif. Geo. 32 (6) 120 (June 1979).
- (6) Datzman, P.A., "The Erosion Hazard Rating System of the Coast Forest District: How Valid is it as a Predictor of Erosion and Can a Better Prediction Equation be Developed?" M.S. Thesis, Humbolt State Univ., Arcata, California (June 1978).
- (7) Nobel, D., "Silver Creek - Field Laboratory in the Idaho Batholith," Forest Resources West 4 (November 1980).
- (8) O'Leary, S., "Silvicultural 208 Activities in California," Presented in Technical Bulletin No. 389 (November 1982).
- (9) Swanson, F., "Cumulative Stream Impacts," Paper presented at Workshop on Stream Management, Oregon State University, Corvallis, Oregon (May 1982).
- (10) "An Approach to Water Resources Evaluation of Nonpoint Silvicultural Sources; A Procedural Handbook," EPA-600/8-80-012 (August 1980).
- (11) Laven, R.D., and Lehre', A.K., "The Effects of Timber Harvest and Roads on Sediment Yield and Channel Morphology in the Fox Planning Unit," USDA Forest Service, Six Rivers National Forest (July 1977).
- (12) Cline, R., et al. "Procedure for Predicting Sediment Yields- A Working Draft," USDA For. Serv., North Region and Inter-mountain Region (No Date).
- (13) Lassen, K.R., and Sidle, R.C., "Erosion and Sedimentation Data Catalog of the Pacific Northwest," USDA For. Serv., Pacific Northwest Region R6-WM-050-1981 (September 1981).
- (14) Howes, S., and Hughes, D., "Descriptions of Available Models and Techniques for Computing On-Site Erosion Losses and Mass Movement, Sediment Delivery Indices, and Erosion Hazard Ratings," Forest Soil Horizons 7, USDA For. Serv., Pacific Northwest Region, Portland, Oregon (May 1981).
- (15) Spiesschaert, D., Carleson, D., Carter, G., Duncan, S., Madison, R., Manson, R., and Pyles, M., "Minimizing Debris Avalanches on Forest Land," A Report to the State Forester, Oregon State Department of Forestry (December 1982).
- (16) "Pollution Control in the Forest Products Industry," EPA 625/3-79-010 Envir. Res. Info. Center, Cincinnati, Ohio (August 1979).
- (17) "Annual Survey of Ongoing Research on the Impact of Forest Management Practices on Water Quality and Utility - 1982," NCASI Technical Bulletin No. 373 (June 1982).

- (18) McGreer, D.J., "A Study of Erosion from Skid Trails in Northern Idaho," in Measuring and Assessing the Effectiveness of Alternative Forest Management Practices on Water Quality, NCASI Technical Bulletin No. 353 (August 1981).
- (19) "Factors Affecting Changes in the Percent of Fine Sediment in Gravel Bedded Channels," NCASI Technical Bulletin No. 354 (August 1981).
- (20) McGreer, D., Forest Hydrologist, Potlatch Corp., Lewiston Idaho, Personal Communication (December 28, 1981).
- (21) Hewlett, J., "Models in Land Use Hydrology", Technical Report No. TR 113, Department of Water Affairs, Forestry, and Environmental Conservation, Pretoria, So. Africa (May, 1980).
- (22) Novotny, V. and Chesters, G., Handbook of Non-Point Source Pollution, Van Nostrand Reinhold, New York (1981).
- (23) "A Study of the Selection, Calibration, and Verification of Mathematical Water Quality Models", NCASI Technical Bulletin No. 367, New York, NY (March, 1982).
- (24) Ambrose, R., et al., "Models for Analyzing Eutrophication in Watersheds-A Selection Methodology", Chesapeake Bay Program, US EPA, Annapolis, MD (1981).

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

DETENTION POND SIZING TO ACHIEVE WATER QUALITY OBJECTIVES

Roger K. Wells, P.E.

HMM Associates, Inc.

Introduction

HMM Associates has recently performed a modeling study of the Black Warrior River in Tuscaloosa County, Alabama. The study area consisted of the reach downstream from the Oliver Lock and Dam to the Warrior Lock and Dam--approximately 75 river miles. The goals of the study were twofold: (1) to perform the necessary water quality modeling to support an application for discharge permit under NPDES and (2) since the reach is water quality-limited during a portion of the year, to demonstrate the adequacy of a proposed discharge detention system. As part of (2), the size of the pond was to be determined.

Overall Method

As part of the effort, several mathematical modeling techniques were employed. The water quality of the reach was modeled using QUAL-II. Runs were made with and without the proposed discharge. QUAL-II was used in the flow augmentation mode to generate data to provide a correlation between river conditions and allowable discharge. Once this relationship was established, a synthetic streamflow model was developed to simulate a realistic sequence of daily flows, and finally a detention pond model was developed to determine the annual maximum storage requirements for a 50-year period of simulation. Figure 1 shows an example of the desired frequency distribution. Extreme value distributions such as this are difficult to construct solely on the basis of existing records because of the distribution's skew. Skew is a measure of the third moment, which is greatly influenced by rare outlying events. Rarely are streamflow records long enough for reliable estimates of the third moment. Using a reliable estimate of this relationship, designers can pick the required pond volume with a knowledge of the return period of conditions which may affect facility capacity. The following sections discuss in turn the analytical steps to develop this curve.

Model Set-Up and Input Parameters

The first step in setting up a QUAL-II simulation is to schematize the river so that important waste loads, tributaries, and geometrical characteristics may be accounted for. For this study, a total of 154 1/2-mile computational elements for the reach between the Oliver Lock and Dam and the Warrior Lock and Dam downstream were used. These computational elements were grouped into reaches, each with its own reaction rates, temperature, and geometry. Fifteen reaches with 11 different hydraulic characteristics were incorporated into the model. Each reach also receives distributed runoff flow. Eleven computational elements were identified as point source discharges or withdrawals. Small tributary flows were considered point sources in this study. Figure 2 shows the river schematic developed for this study.

KΣ PROBABILITY & DIVISIONS
GROUPED & ORDERED

46 8003

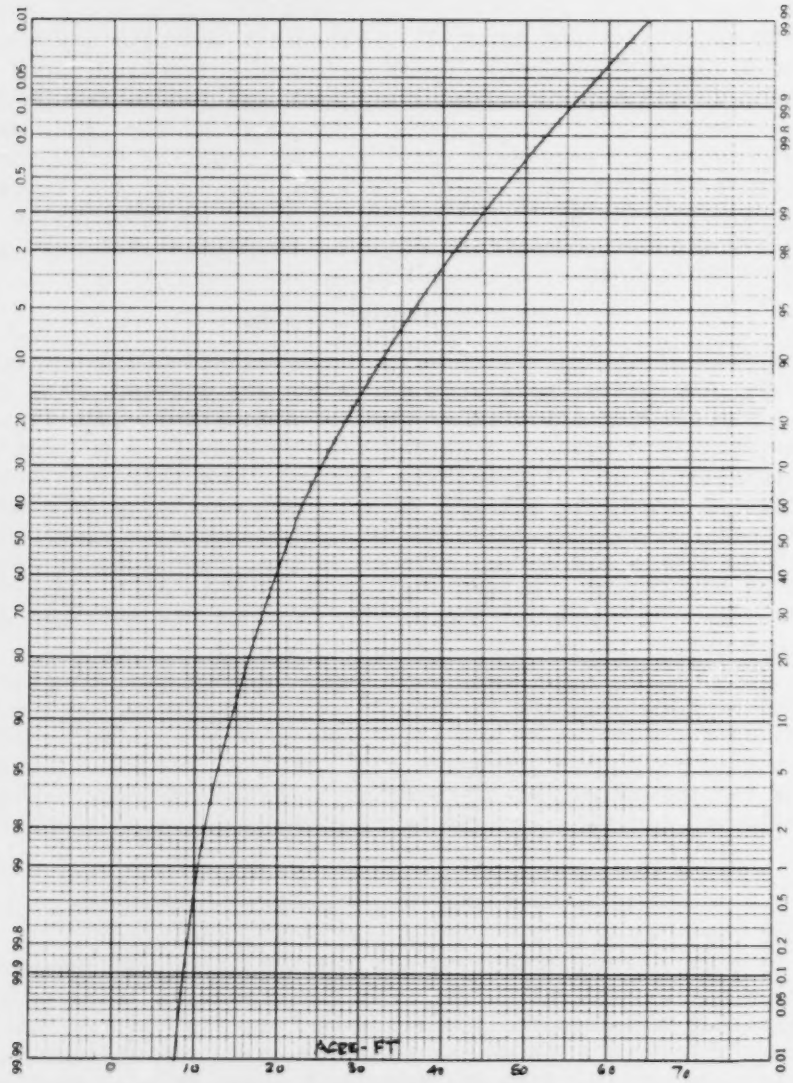


FIG. 1 FREQUENCY DISTRIBUTION FOR ANNUAL MAXIMUM STORAGE

Figure 1

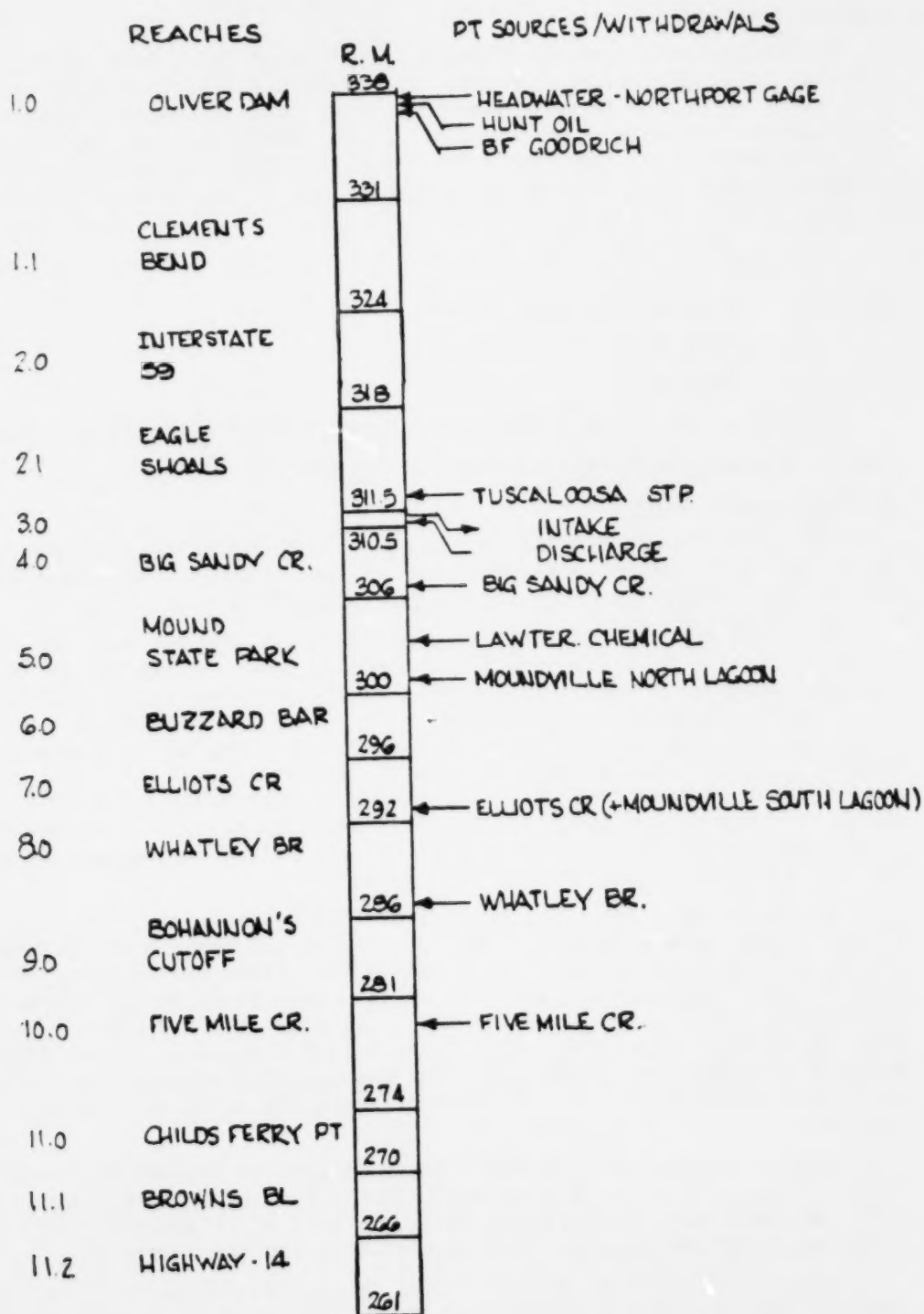


Figure 2
Black Warrior River Model

Hydraulic Data

Hydraulic data were developed as a result of field work which was undertaken in support of this study. The field study is discussed later in this paper. These data were used to develop trapezoidal approximations of each section for model verification. Since different flows were necessary for subsequent model runs, velocity, flow, and depth relationships were derived for later runs. These relationships are of the following form:

$$V = aQ^b$$

$$D = \alpha Q^\beta$$

where V = stream velocity, in ft/sec;

Q = stream flow, in ft³/sec;

D = stream depth, in ft; and

a , b , α , and β are coefficients determined as part of this study.

Table 1 summarizes the coefficients so determined.

Reaction Coefficients

The reaction coefficients which must be specified include the carbonaceous BOD reaction rate (K_1), the NH_3 oxidation rate (CKNH_3), the NO_2 oxidation rate (CKNO_2), and the atmospheric reaeration rate (K_2).

K_1 . The BOD reaction rate is a parameter which expresses the rate (in units of days⁻¹) of oxidation of carbonaceous BOD. During the field measurement program, a sample was taken near the proposed discharge location for a 30-day BOD analysis. These data were analyzed for reaction rate using a non-linear least squares regression technique known as the Gauss-Newton method. Figure 3 summarizes the results of this analysis. It can be seen that the oxygen demand is still increasing substantially up to 30 days, which indicates a low reaction rate. It was also felt that the point at five days might be invalid because of its low value, so regressions were performed both with and without this value.

In selecting a reaction coefficient (K_1) for modeling, the values from Figure 3 were used as limits. The rate was varied until a balance was struck between measured BODs and measured DOs within this range. The final rate selected for modeling was 0.02 day⁻¹. This value is considerably below the values traditionally used for modeling, i.e., 0.1-0.2 day⁻¹. Since this test is the only known long-term BOD test from this reach of the Black Warrior River and reasonable agreement was obtained between field measurements and calculations, it was decided to use the value of 0.02 day⁻¹ for modeling.

CKNH_3 . A value of 0.05 day⁻¹ was selected for the ammonia oxidation rate. This is based on discussions with the regulatory authorities and reports of their work in the Black Warrior River, which indicated that nitrification is not an important oxygen-consuming process in-stream.

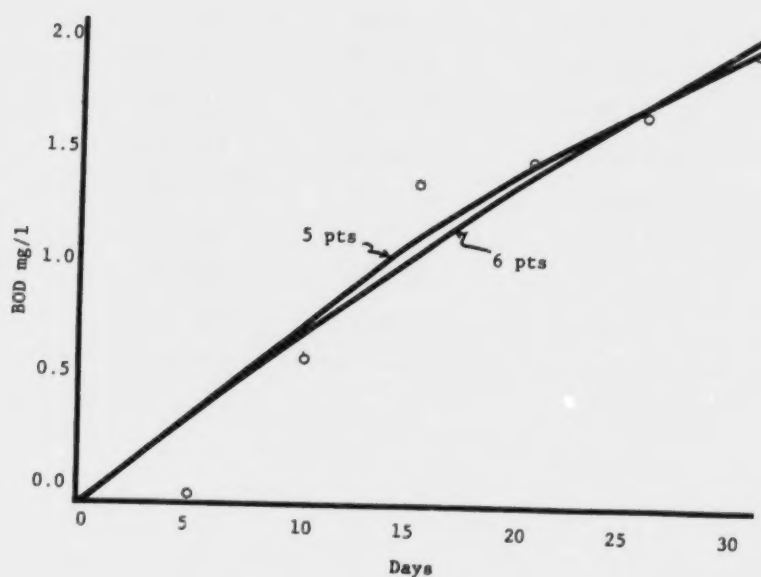
CKNO_2 . The nitrite oxidation rate chosen was 2.0 day⁻¹. This corresponds to the idea that nitrite is merely a short-lived intermediate step in the process of oxidizing NH_3 to NO_3 . Without detailed nitrogen measurements to support the use of another value, the upper end of the range suggested in the QUAL-II user's manual was chosen.

Table 1
Stream Flow Coefficients

Reach Number	a	b	α	β
1.0	3.408E-4	0.929	11.085	0.0523
1.1	3.408E-4	0.929	11.085	0.0523
2.0	1.943E-4	0.956	20.407	0.0329
2.1	1.943E-4	0.956	20.407	0.0329
3.0	3.840E-4	0.904	10.054	0.0561
4.0	3.155E-4	0.934	11.368	0.0514
5.0	3.417E-4	0.931	8.756	0.0617
6.0	2.538E-4	0.949	16.817	0.0383
7.0	3.203E-4	0.933	9.496	0.0584
8.0	1.953E-4	0.949	21.187	0.0320
9.0	2.072E-4	0.946	19.240	0.0345
10.0	1.472E-4	0.952	27.066	0.0261
11.0	1.314E-4	0.968	29.237	0.0245
11.1	1.314E-4	0.968	29.237	0.0245
11.2	1.314E-4	0.968	29.237	0.0245

30-Day BOD

Time (days)	BOD (mg/l)
5	0.05
10	0.65
15	1.45
20	1.55
25	1.75
30	2.05



Data Base	Reaction Rate	Corr. Coeff.
6-point	0.0118	0.91517
5-point	0.0260	0.90644

Figure 3

Reaeration Rate

Atmospheric reaeration is the fundamental source to replenish in-stream DO. Unfortunately, a wide range of estimates of the coefficient is available. The method of Tsvoglou and Wallace was used for the calibration/verification, a method recommended by EPA in the QUAL-II documentation. The form of this estimate is:

$$K_2 = 3600 K Se U$$

where K_2 = reaeration rate, in hour^{-1} ;

K = a constant $\approx 0.05\text{-}0.1 \text{ ft}^{-1}$;

Se = energy gradient slope, in ft/ft ; and

U = stream velocity, in ft/sec .

It can be seen that K_2 depends on the energy gradient slope. At low flows, the Warrior Pool becomes more lake-like, with a near-zero energy gradient, and a method more appropriate for a lake environment is used. Therefore, it was decided that a method outlined by Brown* would be used. This method relates the surface transfer coefficient K_L to the wind speed:

$$K_L = 0.05 w^2$$

where K_L = surface transfer coefficient, in m/day ; and

w = wind speed, in m/sec .

For the low flow portion of this study, K_L was estimated to be $0.25 \text{ m}/\text{day}$ ($0.82 \text{ ft}/\text{day}$), based on a 5 mph wind speed. Table 2 summarizes the K_2 selected for each reach for the low flow runs.

Initial Conditions

For a steady-state simulation, the only parameter which needs to be specified is the temperature since it affects the adjusted reaction coefficient and the saturation value of DO. A value of 85.1°F (29.5°C) was chosen. This value is the mean August temperature based on daily measurements at Northport, Alabama, during the most recently published water resources data period (USGS, as of June 1982).

Headwater Sources

The only headwater considered in this study is the Black Warrior River at the Oliver Lock and Dam. The USGS in Tuscaloosa was contacted to determine the most recent estimate of the $7Q_{10}$. This parameter is significantly affected by the operating policy of the upstream dams. The current operating mode was instituted in 1964. The values of $7Q_{10}$ for this period were obtained from the USGS. The value 605.7 cfs is used as the starting point for all low flow modeling. As mentioned before, in-stream temperature is taken to be 85.1°F , and the headwater DO level is taken to be the saturation value at 85.1°F , or $7.4 \text{ mg}/\text{l}$. The value of BOD_5 at the headwaters is taken to be $0.45 \text{ mg}/\text{l}$. The source of this estimate consists of measurements taken during the field study conducted by HMM as part of this study and measure-

*Brown, Russ T. Modeling the Effects of Wind Reaeration. Report No. WR28-2-520-117. Tennessee Valley Authority. March 1981.

Table 2
Reaeration Values (K_2) for Each Reach, Based on 5 mph Wind Speed

Reach Number	Depth (ft)	K_2 (day ⁻¹)
1,0, 1.1	15.5	0.053
2.0, 2.1	25.2	0.033
3.0	14.4	0.057
4.0	15.8	0.052
5.0	13.0	0.063
6.0	21.5	0.038
7.0	13.8	0.059
8.0	26.0	0.032
9.0	24.0	0.034
10.0	32.0	0.026
11.0, 11.1, 11.2	34.2	0.024

ments provided by AWIC. Table 3 summarizes these data, as well as measurements of NH_3 , NO_3 , and PO_4 (1980 only) included in 1980 and 1981 studies by AWIC. The mean value of BOD_5 , NO_3 , and TSS was included in the headwater concentration levels for these constituents. The concentration of NH_3 was entered as zero, and PO_4 was not modeled since all measurements were below the detection limit.

The QUAL-II model provides for a complete description of the relationship among algae, nutrients, and oxygen. In the field measurement programs conducted by AWIC in September 1980 and in June-July 1981, numerous oxygen samples were taken that indicated that the water was supersaturated with oxygen. This also occurred in the HMM measurements taken in August 1982. These findings indicate that a net oxygen credit, due to photosynthesis by algae, is occurring, since this is assumed to be the only process producing oxygen in-stream. In August 1981 AWIC indicated, in their analysis concerning the Hunt Oil Company expansion, that a net contribution of 0.15 mg/l/day of oxygen was being made by algal processes. This rate was incorporated into the HMM modeling effort. The QUAL-II model couples algal growth to the availability of nutrients and light. It also allows for creating $\text{NH}_3\text{-N}$ from algae through a coefficient which represents the fraction of respired algal biomass which is resolubilized as $\text{NH}_3\text{-N}$ by bacterial action. Unless the reaction rate for $\text{NH}_3\text{-N}$ to $\text{NO}_2\text{-N}$ is very low, a net oxygen deficit will result from these coupled processes. Thus, the supersaturated levels of oxygen also support the idea that little or no nitrification is occurring in-stream, an opinion expressed by AWIC when evaluating the Hunt Oil expansion. In QUAL-II, the presence of algae is considered to be represented by the measurement of chlorophyll A. This was input to the model at 20 $\mu\text{g/l}$, which yields an oxygen production rate of 0.15 mg/l/day at the headwaters. This value rises slightly until about river mile 325, when increasing river depths limit the availability of light for algal growth, and the downstream chlorophyll concentration drops to about 5.25 $\mu\text{g/l}$ at the Warrior Dam, yielding an oxygen production rate of 0.04 mg/l/day.

Existing Point Source Loads

Point source loadings in QUAL-II include unmodeled tributaries as well as industrial and municipal discharges. Sources of data on existing discharges include the Tuscaloosa County 208 Report and AWIC. Table 4 summarizes the characteristics of all modeled discharges.

New Discharge

The new discharge is subject to new source performance standards. The following parameters summarize the discharge's characteristics.

$$Q_D = 5.028 \text{ cfs};$$

$$\text{TSS} = 201 \text{ mg/l};$$

$$\text{BOD}_5 = 84.7 \text{ mg/l}; \text{ and}$$

$$\text{BOD}_u/\text{BOD}_5 = 2.62$$

Field Measurement Program

On August 3-4, 1982, HMM Associates undertook a field measurement program on the Black Warrior River to obtain data necessary to characterize the

Table 3
BOD₅, NO₃, NH₃, and PO₄ Near the Headwaters

Date	TSS	BOD ₅	River Mile	Source	NO ₃	NH ₃	PO ₄
8/82	8.4	0.05	311	Field Study	-	-	-
	3.6	0.15	320	Field Study	-	-	-
	4.4	0.25	329	Field Study	-	-	-
6-7/81	-	1.3	337	AWIC	0.92	< 1	-
	-	0.3	337	AWIC	0.74	< 1	-
	-	-	337	AWIC	0.54	< 1	-
9/80	6	0.4	337	AWIC	0.43	1	-
	5	0.5	337	AWIC	0.45	-	-
	6	0.4	337	AWIC	0.44	-	-
	3	0.5	337	AWIC	0.42	-	< .2
	4	0.4	337	AWIC	0.41	-	< .2
	11	0.7	337	AWIC	0.37	-	< .2
Mean =					5.71	0.45	0.524
Std. Dev. =					2.56	0.33	0.185

Table 4
Summary of Point Sources

Source Name	Flow (cfs)	Temp. (°F)	DO (mg/l)	BOD _u (mg/l)	TSS (mg/l)	NH ₃ (mg/l)
Hunt Oil	0.5 (2)	95 (3)	5.4 (2)	143 (1)	79.3 (1)	768(1)
B. F. Goodrich	1.39 (2)	95 (3)	5.4 (2)	7.3 (2)	0.0	0
Tuscaloosa STP	24.0 (2)	85.1	5.4 (2)	43.8 (2)	30.0 (3)	20(2)
Big Sandy Creek	34.0 (5)	75.2(4)	7.2 (4)	1.58(4)	24.0 (4)	-
Lawter Chemical	0.18	85.1	5.0	123.0 (1)	134 (1)	
Moundville N./Lagoon	0.186(1)	85.1	5.4	65.7 (1)	135 (1)	
Elliot's Creek/Moundville S.	7.893(1,5)	85.1	7.38(6)	5.46	1.59(4)	
Whatley Branch	0.1 (5)	85.1	7.45	0.0 (4)	3.0 (4)	
Five Mile Creek	2.8 (5)	85.1	7.45	1.58(4)	37.3 (4)	

Note: Numbers in parentheses above indicate references below:

- (1) AWIC permit.
- (2) AWIC memo.
- (3) Tuscaloosa County 208 Report.
- (4) Field study measurement, August 3-4, 1982.
- (5) GSA Bulletin 113.
- (6) By calculation.

hydraulic characteristics of the river, to measure depths in detail in the vicinity of the intake and discharge areas, and to obtain DO, BOD, TSS, and color samples throughout the modeled reach of the river. Samples were also taken from Big Sandy Creek, Elliot's Creek (upstream of the Moundville South Lagoon), Whatley Branch, and Five Mile Creek.

The hydraulic characteristics were determined by bathymetric measurements taken in the river with a Raytheon DE719B recording fathometer. The daily discharges at the Northport Gage for the two days of the study were obtained from the USGS. The flow for August 3 was 6530 cfs, and the flow for August 4 was 5790 cfs. In modeling, the mean flow of 6160 cfs was used, since the measurements of DO and BOD spanned the two days. The water quality parameters were sampled at each bathymetric measurement point. In situ DO and temperature measurements were taken with a Yellow Springs Model 54B DO meter. Table 5 summarizes the samples taken during the field study.

Modeling Results

Calibration/Verification

The QUAL-II model was set up as described with the measured hydraulic characteristics as input. Using the data measured on August 3-4 as a reference, the model was run to verify that reasonable results were produced. Minor adjustments were made in reaction rates and the method of treating the hydraulics until reasonable agreement was reached between calculated and measured DO and BOD. The results of this process are shown in Figure 4. It should be noted that the City of Tuscaloosa Sewage Treatment Plant (STP) was not operating properly on the days of measurement. The level of BOD₅ in the STP effluent was at an average level of 211 mg/l. The level of NH₃ assumed in the STP discharge was the permit level of NH₃ scaled by the ratio of actual BOD₅ to permit BOD₅:

$$\text{NH}_3 \text{ discharge} = 20 \times 211/30 = 141 \text{ mg/l}$$

It can be seen that reasonable agreement was obtained, particularly in the case of DO. The large amount of scatter in the measured BODs can be accounted for by incomplete mixing of the STP discharge in the portion of the river where the greatest changes in BOD levels would be expected.

No Mill

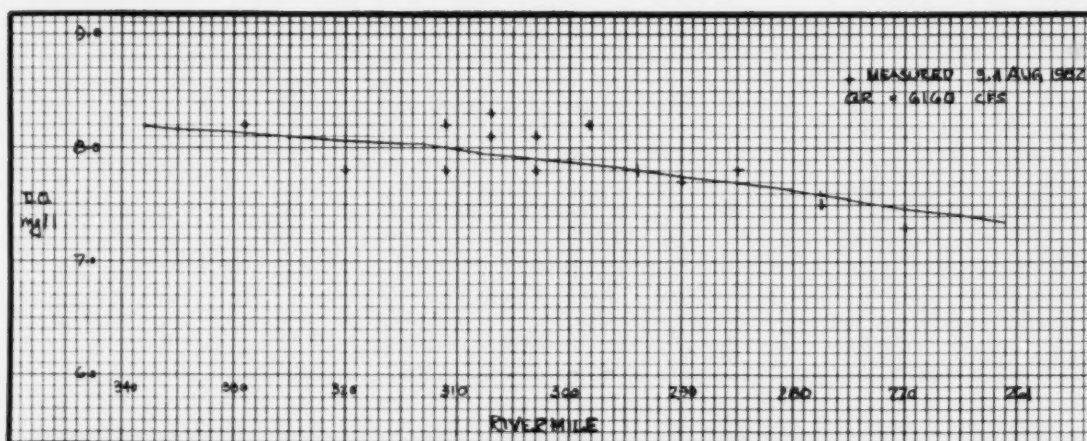
The model was next run with the river flow at the 7Q₁₀ level of 605.7 cfs and all present dischargers operating at their full permit limits. The minimum calculated DO was 5.39 mg/l, occurring at the downstream end of the reach.

With New Discharge

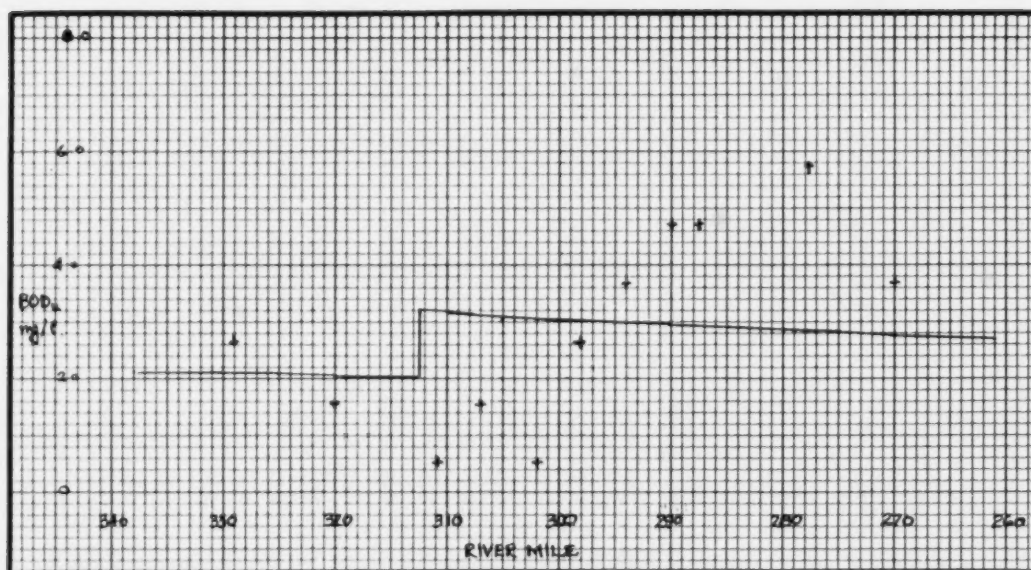
When the new discharge was input at the levels described above and the river flow was 605.7 cfs, the flow augmentation option was used. In this case, a total augmentation of 60.9 cfs was required to maintain 5.0 mg/l of DO. Several runs were made to provide information necessary for input to the retention plan analysis. Since the maximum daily discharge permitted under NSPS is twice the 30-day average, this level is considered to be the upper limit for the discharge. Computer runs were made at 25%, 50%, 75%, 150%, and 200% of the 30-day average discharge levels to determine the critical flows for these discharge levels. The results are plotted in Figure 5,

Table 5
Field Study Sample Summary

Location	Date:Time	DO (mg/l)	Temp. (°C)	BOD ₅ (mg/l)	TSS (mg/l)	Color APHA Apparent	Turbidity (NTU)	Cross- section (Y/N)
Elliot's Creek @ S.R. 69	8/2:1825	6.6	25	0.05	8.0	60	10.0	N
Five Mile Creek @ S.R. 60	8/2:1920	7.0	24	0.15	37.3	200	5.9	N
Whatley Branch @ S.R. 69	8/2:1934	6.6	25	0.00	3.0	50	9.5	N
Big Sandy Creek @ S.R. 69	8/2:1915	7.2	24	0.15	24.0	125	32.0	N
R.M. 311	8/3:1520	7.8	29	0.05	8.4	15	7.8	Y
R.M. 320	8/3:1611	7.8	29.2	0.15	3.6	15	6.2	Y
R.M. 329	8/3:1656	8.2	29.2	0.25	4.4	20	7.0	Y
R.M. 306.9	8/3:1748	8.3	29	0.15	2.4	10	7.5	Y
R.M. 303	8/4:1015	7.8	29	0.05	1.2	15	5.2	Y
R.M. 298.1	8/4:1036	8.2	29	0.25	10.0	20	6.2	Y
R.M. 294	8/4:1056	7.8	29	0.35	3.6	20	6.6	Y
R.M. 290	8/4:1127	7.7	29	0.45	1.6	15	5.9	Y
R.M. 285	8/4:1152	7.8	29	0.45	-	-	-	Y
R.M. 277.6	8/4:1223	7.5	28.8	0.55	-	-	-	Y
R.M. 270	8/4:1257	7.3	28.6	0.35	-	-	-	Y
R.M. 303.7	8/4:1435	8.1	29	-	3.6	15	5.9	N
R.M. 307 (Rt)	8/4:1449	8.1	29	-	4.0	15	6.2	N
R.M. 311	8/4:1503	8.2	29	-	1.6	20	6.6	N



DO Calibration



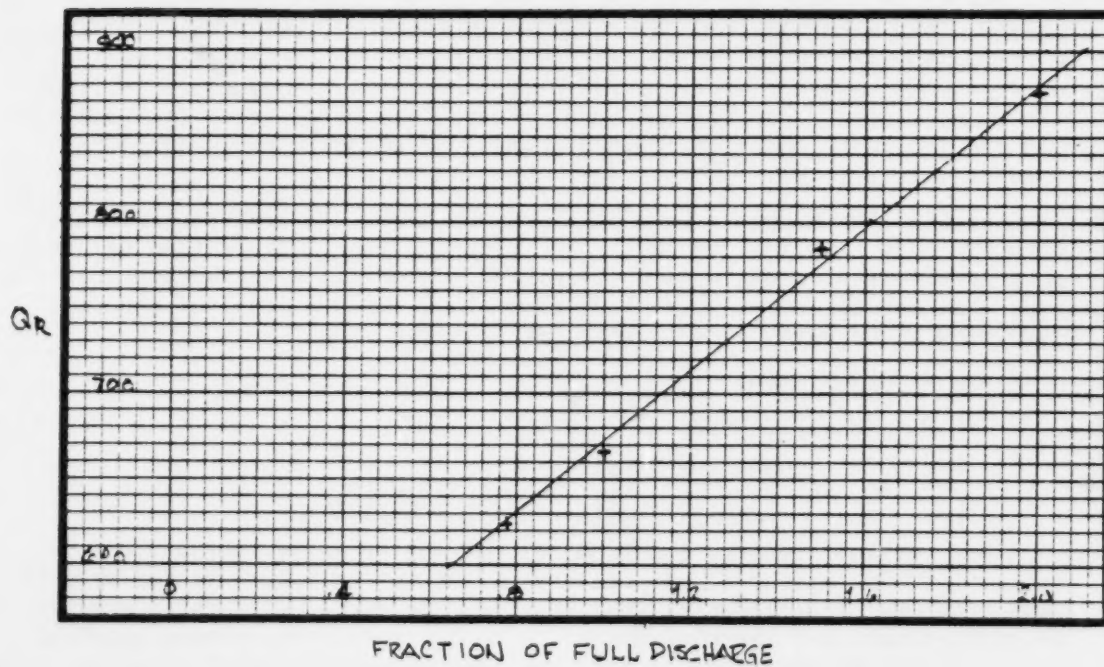
BOD Calibration

Figure 4

Mill Discharge Fraction vs. River Flow

River Flow (cfs)	Discharge Volume (%)
465.0	0
623.2	75
666.6	100
786.0	150
877.0	200

Note: 100% discharge is 5.028 cfs and 84.7 mg/l BOD₅ or 2300 lbs/day.



Regression Analysis of Critical River Flows

Figure 5

and a regression line was calculated for use in subsequent analyses. The equation of the regression line is given by:

$$\begin{aligned} X &= 2.0 & 881 < Q \\ X &= -2.229 + 0.004798 \times Q & 465 < Q < 881 \\ X &= 0 & \text{otherwise} \end{aligned}$$

where X = fraction of maximum new discharge; and

Q = river flow.

The correlation coefficient is 0.9982.

Persistence of Daily Flows

It is evident from the results reported in the previous sections that the allowable discharge level is highly dependent on daily river flow values. Since the current river operating system went into effect in the early 1960's, the daily flow fluctuations have been greatly reduced, and prolonged periods of low flow are rare. This is evident in the fact that for the period of record of the Northport Gage (1895-present), the 10-year 7-day low flow ($7Q_{10}$) has risen from approximately 96 cfs to 605.7 cfs. Since the volume of retention is greatly dependent on day-to-day flow variations, an analysis has been performed to study the actual persistence of day-to-day flows. As an example of the issue at hand, the $7Q_{10}$ is 605.7. This could be taken to mean that for at least 7 days, no discharge could occur (if the limiting flow happened to be at or above this value), resulting in 7 days of required storage. However, this 7-day average could also be the result of the following sequence of flows (in cfs): 248, 248, 1500, 248, 248, 1500, 248--resulting in 2 days of required storage (if the limiting flow was less than 1500). Thus, modeling the day-to-day variation is fundamental to studying this problem.

The USGS has supplied statistics on the daily flow records from the Northport Gage from 1964 to 1981. The distribution of daily flow values is shown in Figure 6, along with a calculated log normal distribution which shows that the log normal distribution is a reasonable model of the daily river flow. The parameters of this distribution are:

$$\begin{aligned} \overline{\log Q} &= 8.31 \quad (\text{mean}) \\ V_Q &= 1.30 \quad (\text{variance}) \\ N &= 6575 \quad (\text{number of samples}) \end{aligned}$$

Synthetic Stream Flow Generation

This section presents a method for generating daily stream flows by computer. These stream flows will correspond to the actual stream flows on three measures of performance--the mean and the variance of the logarithm of the daily flows, and the mean annual 7-day low flow. The method used has been described by Loucks *et al.** A simple autoregressive model is used. This type of model is a first-order Markov process which recognizes that the flow

*Loucks, D. P., J. R. Stedinger, and D. A. Haith. Water Resource Systems Planning and Analysis. Prentice-Hall. 1981. Chapter 6.

46 8003

K-E PROBABILITY & SO DIVISIONS
ALPHAL & ESSER CO. NEW YORK

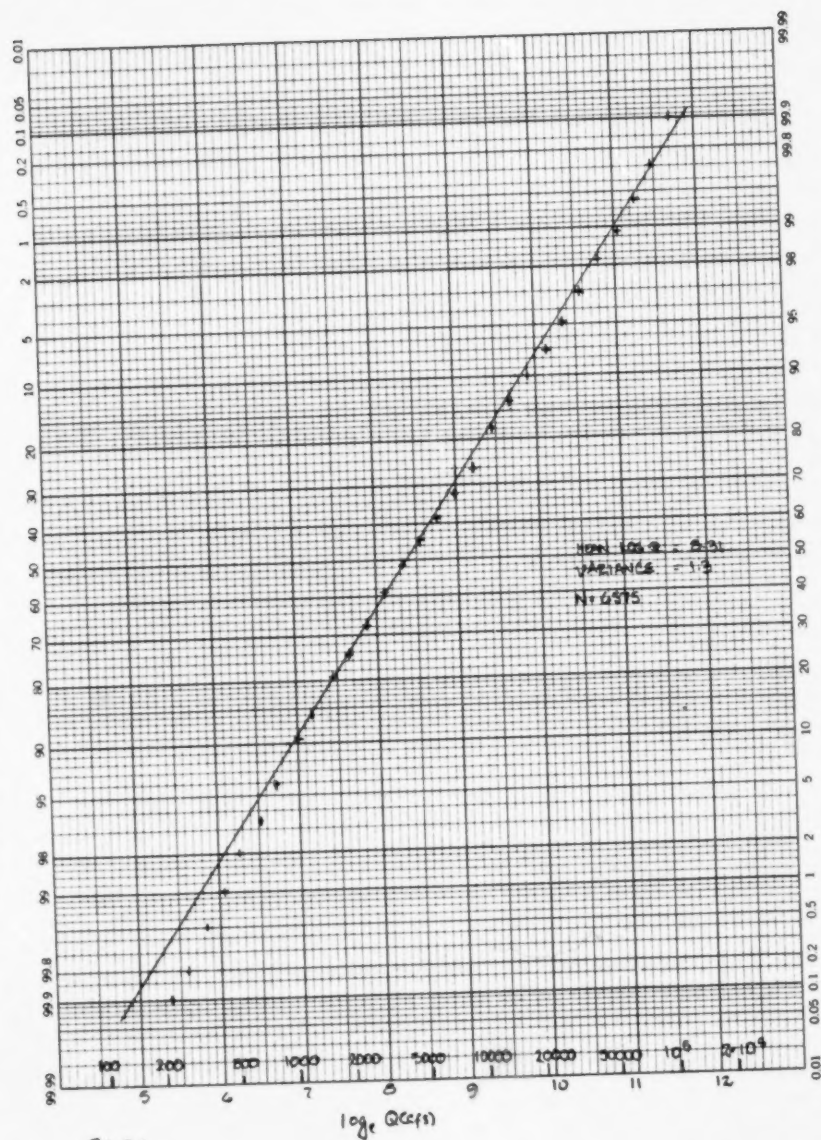


Fig 5-1

Figure 6
Distribution of Daily Flow Values

today is to some degree dependent on the flow yesterday. The generation of synthetic stream flows requires the marginal distribution of flow on one day given the flow on the previous day. Thus, if there is a low flow today, a low flow is more likely than a high flow tomorrow. The preferred method of generating such flows is to transform the flows to a normally distributed random variable. In this case, this corresponds to dealing with the logarithms of the flows. A first-order Markov model for such a variable is given by:

$$X_{t+1} = \mu + \rho_X(X_t - \mu) + V_t \sigma_X \sqrt{1 - \rho_X^2}$$

where $X = \log Q$;

μ = mean of X ;

ρ_X = lag 1 autocorrelation coefficient;

t = time, in -days;

σ_X = standard deviation of X ; and

V_t = unit normal random variate.

In this case, $\mu = 8.31$;

$$\sigma_X = \sqrt{1.3} = 1.14; \text{ and}$$

ρ is varied until the mean annual 7-day low flow is reproduced.

Table 6 summarizes a number of computer runs for various values of ρ . For this study, a value of 0.45 is used.

Retention Pond Model

The basic mathematical expression of a detention basin is given by the following ordinary differential equation:

$$\frac{ds}{dt} = I(t) - O(t) \quad (1)$$

where s = storage volume;

I = inflow;

O = outflow; and

t = time.

In this case, I is a constant 3.25 mgd. The problem becomes one of determining $O(t)$. The following procedure is used. The flow generation model described above is used to generate a daily flow value. The relationship determined previously is used to determine the maximum permissible discharge:

$f = 0$	$Q_R < 465$
$f = 0.00478Q - 2.229$	$465 < Q_R < 881$
$f = 2.0$	$Q_R > 881$

where f = fraction of daily average discharge (3.25 mgd); and

Q_R = river flow, in cfs.

Table 6
Dependence of 7-Day Low Flows on the Lag 1 Autocorrelation Coefficient

ρ	Mean 7-Day Low Flow
0.3	1364
0.4	1146
0.5	959
0.6	806
0.7	695
0.8	631
0.9	600
0.45	1050.3

USGS value is 1058.11.

Equation (1), in discrete form and rearranged, is used to compute the daily change in required storage:

$$\Delta S = [I - O] \Delta t$$

If storage goes negative, it is set equal to zero. Another flow is generated, and the process repeats. A flow chart representing this procedure is shown in Figure 7.

An algorithm was written to implement this procedure on a small computer. A run modeling performance for 50 years was made and the resulting distribution of annual extreme maximum storage volumes plotted. Table 7 contains the results of this run. A Pearson Type III extreme value probability distribution was fitted to these observations. Using this distribution, a graph was constructed to show the required storage volumes for different return periods (Figure 8).

Proposed Control Plan

Since development of this plan has been based on the distribution of low river flows, it is proposed that the level of discharge be controlled in coordination with present flow levels. In order to do this, river flow must be measured or estimated locally. Using the measured flow level, the discharge may be regulated according to the following relationship:

$$\begin{array}{ll} Q_D = 0 & Q_R \leq 465 \\ Q_D = 24.12E-3 \times Q_R - 11.21 & 465 < Q_R < 881 \\ Q_D = 10.06 & 881 < Q_R \end{array}$$

where Q_D = discharge flow, in cfs; and

Q_R = river flow, in cfs.

The purpose of the discharge control plan is to maintain Black Warrior water quality while at the same time allowing the facility to operate at design capacity. To this end, retention storage has been incorporated into the overall facility design.

Conclusion

A method of generating a time series for the modeling of extreme values, in this case annual maximum storage volumes, has been presented. The dependence of these processes on higher moments of the underlying distribution makes use of existing records unreliable, since rarely are hydrological time sequences long enough for the estimation of higher-order moments. In the future, models using higher-order autoregressive and/or moving average parameters may be fruitfully explored.

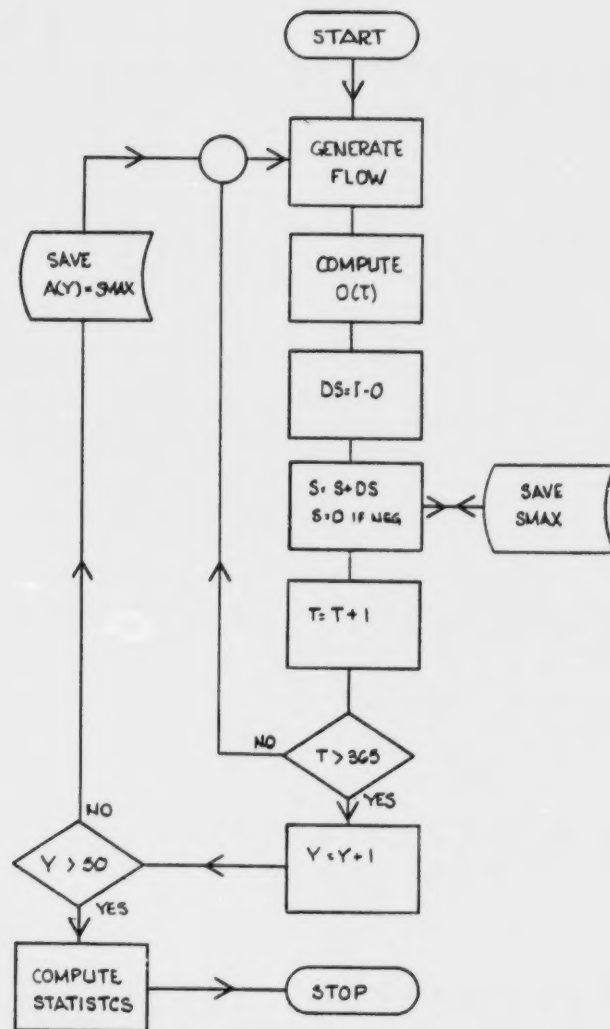


Figure 7

Flow Chart for Retention Pond Performance Computer Program

Table 7. Annual Maximum Storage for 50 Years

Year	Acre-Feet	Ranked	Plotting Position
1	19.945	9.972	.01
2	19.945	9.972	.03
3	15.040	9.972	.05
4	19.945	9.972	.07
5	26.424	12.197	.09
6	32.327	13.578	.11
7	13.578	14.084	.13
8	15.586	14.382	.15
9	29.056	15.040	.17
10	19.945	15.586	.19
11	9.972	15.731	.21
12	18.258	16.661	.23
13	19.945	17.442	.25
14	43.583	18.258	.27
15	29.918	19.041	.29
16	23.329	19.411	.31
17	31.914	19.945	.33
18	19.945	19.945	.35
19	31.028	19.945	.37
20	12.197	19.945	.39
21	16.661	19.945	.41
22	29.918	19.945	.43
23	14.382	19.945	.45
24	19.945	19.945	.47
25	20.056	19.945	.49
26	19.945	19.945	.51
27	19.945	19.945	.53
28	29.918	19.945	.55
29	19.945	19.945	.57
30	25.755	19.945	.59

Year	Acre-Feet	Ranked	Plotting Position
31	9.972	20.056	.61
32	19.945	21.866	.63
33	19.411	22.525	.65
34	29.940	22.809	.67
35	14.084	23.329	.69
36	15.731	24.658	.71
37	19.945	25.755	.73
38	22.809	26.424	.75
39	19.945	29.056	.77
40	21.866	29.918	.79
41	24.658	29.918	.81
42	30.365	29.918	.83
43	29.918	29.918	.85
44	22.525	29.940	.87
45	45.374	30.365	.89
46	19.041	31.028	.91
47	9.972	31.914	.93
48	19.945	32.327	.95
49	17.442	43.583	.97
50	9.972	45.374	.99

Mean Storage	= 21.824842
Variance	= 59.407978
Third Moment	= 425.11451
Percentage of Time Empty	= 92.394521

Storage Requirements for Various Return Periods		
Storage Volume (acre-feet)	Days at 3.25 mgd	Return Period (years)
27.8	2.79	5
32.2	3.23	10
41.5	4.16	50
44.8	4.49	100

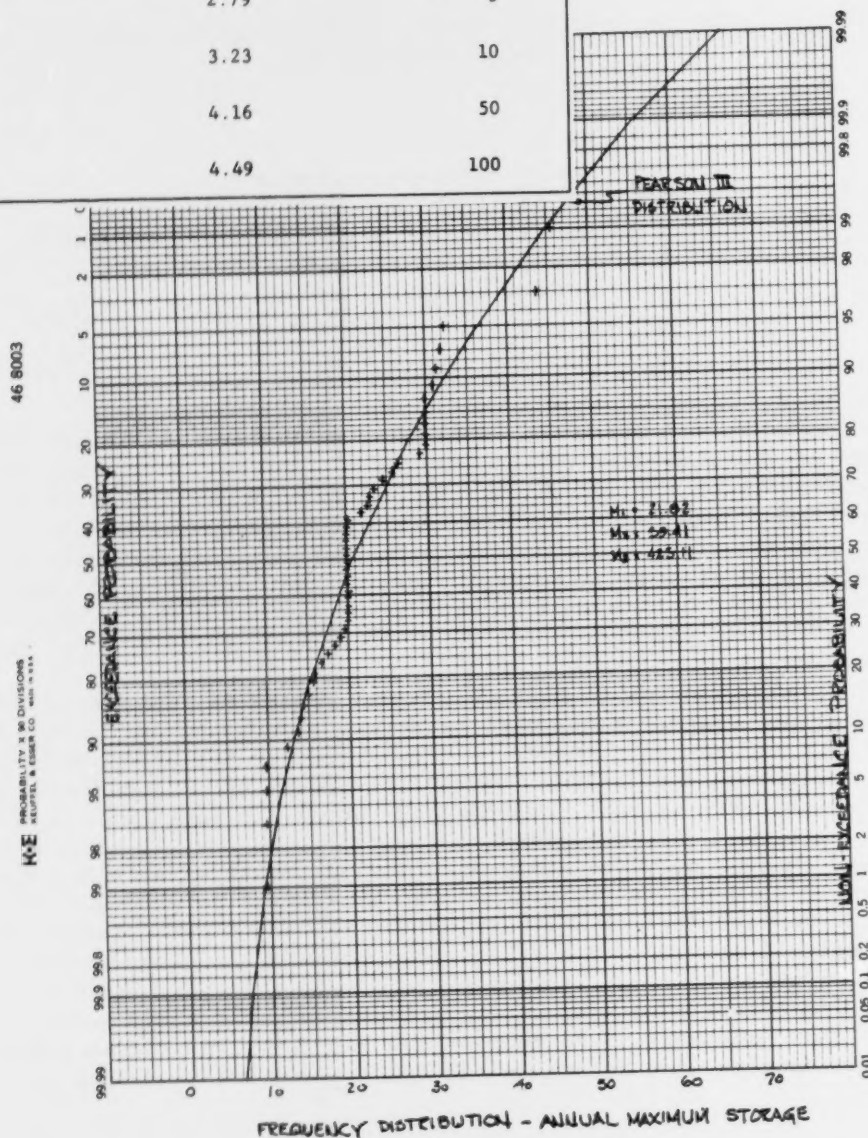


Figure 8

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

A MECHANISTIC SIMULATION FOR TRANSPORT OF NONPOINT
SOURCE POLLUTANTS

By

Daryl B. Simons, President
Ruh-Ming Li, Executive Vice President
Kenneth G. Eggert, Senior Water Resources Engineer
Simons, Li & Associates, Inc.
Fort Collins, Colorado

ABSTRACT

Nonpoint source pollution represents an integration of the behavioral response of the ecosystem to the impacts of land use activities on individual components of the ecosystem. By analyzing basic ecosystem processes and impacts of land use activities on specific processes governing production and transport of nonpoint source pollution, it is possible to predict cause-effect relationships between these activities and water quality. In order to accomplish this goal, a simulation model for evaluation of alternative management practices as a function of both environmental and management parameters is presented.

The simulation may be used to predict watershed response to land use including both planned management practices and unplanned activities associated with catastrophic events. Nonpoint source pollution from these activities and events may include loading of streams by: 1) sediment from surface erosion, mass wasting, and channel bottom and bank erosion, 2) thermal energy, 3) biological contaminants, 4) organic debris, 5) nutrients and dissolved solids, 6) pesticides, and 7) other waste materials either in solution or adsorbed to sediments. The simulation is composed of the following components: 1) a soil-plant-atmospheric water simulation for adjusting soil moisture as a function of evaporation, evapotranspiration, soil water hydraulics, and snowmelt, 2) a kinematic wave surface water routing component, 3) a hydraulically-based sediment yield model, 4) a subsurface flow component, 5) a temperature and dissolved oxygen component, 6) pollutant routing, and 7) streambank erosion and forest litter routing routines.

INTRODUCTION

The theoretical foundation of the watershed simulation component of the generalized planning model for evaluation of alternative management practices is described in this paper. The model is a cause-effect simulation based on mathematical descriptions of watershed processes related to

nonpoint source pollution. Nonpoint source pollution represents an integration of the behavioral response of the ecosystem to the impacts of activities on individual components of the ecosystem. Management activities include both planned management practices, such as road construction, watershed management, stream improvement, low head impoundments, application of pesticides and fertilizers, foliage production and grazing, mechanical site preparation, and disposal of wastes by land treatment, and unplanned activities associated with catastrophic events such as large runoff events. The model as originally formulated was addressed to management activities involving timber harvest, timber planting and replacement, grazing, mechanical site preparation and prescribed fire. However, the mechanistic basis of the model makes it applicable to nearly any watershed activity. Specific applications to agricultural activities as well as waste land treatment and disposal are well within the scope of the model formulation.

The watershed process model is structured in modular form. The modules contain related physical process simulations and are described individually later in this paper. The basic structure of the model is shown in Figures 1 through 3. The model structure was designed to allow a high level of representation of physical processes but still remain within the range of memory constraints normally encountered at most computing centers. The modular nature of the model structure also facilitates substitution of new or improved simulation components as they are developed without extensive revision of the code. The approach has both advantages and disadvantages. The basic advantages are the modular nature as described above, and the capability of using simulation routines that are not overly limited by computer storage. The disadvantage of the structure is that input/output sequences and file management are machine specific and must be tailored to the host computer. However, the basic file management functions used are generally available at most computing centers. Presently, the model has been installed on CDC Cyber 172, HP 1000 and IBM 370 systems. Therefore, the advantage of a more precise representation of physical processes is felt to outweigh any disadvantages due to machine specific job control language.

As shown in Figures 1 through 3, the basic model components are as follows:

1. Water balance
2. Water and sediment routing
3. Pollutant routing
4. Heat and dissolved oxygen routing
5. Streambank erosion
6. Vegetative litter routing
7. Sequential GOAL Programming

WATER BALANCE

Since infiltration exerts a fundamental control on the storm water runoff hydrograph, any long-term hydrologic simulation must have a component for calculating the changes in soil moisture content as a function of time. The primary processes affecting the amount of soil moisture are infiltration, percolation, evaporation, evapotranspiration and drainage. These

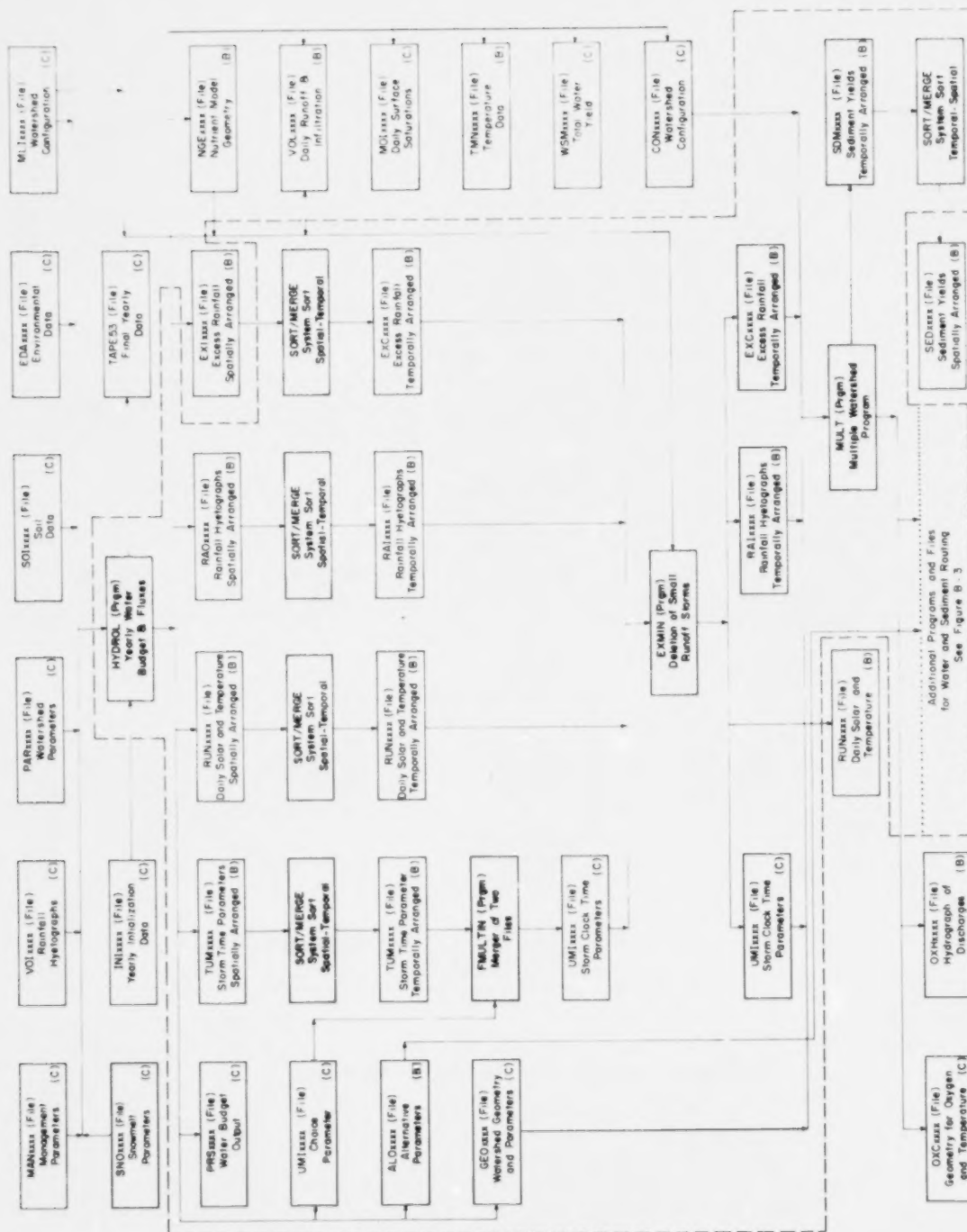


Figure 2. Water budget and flux.

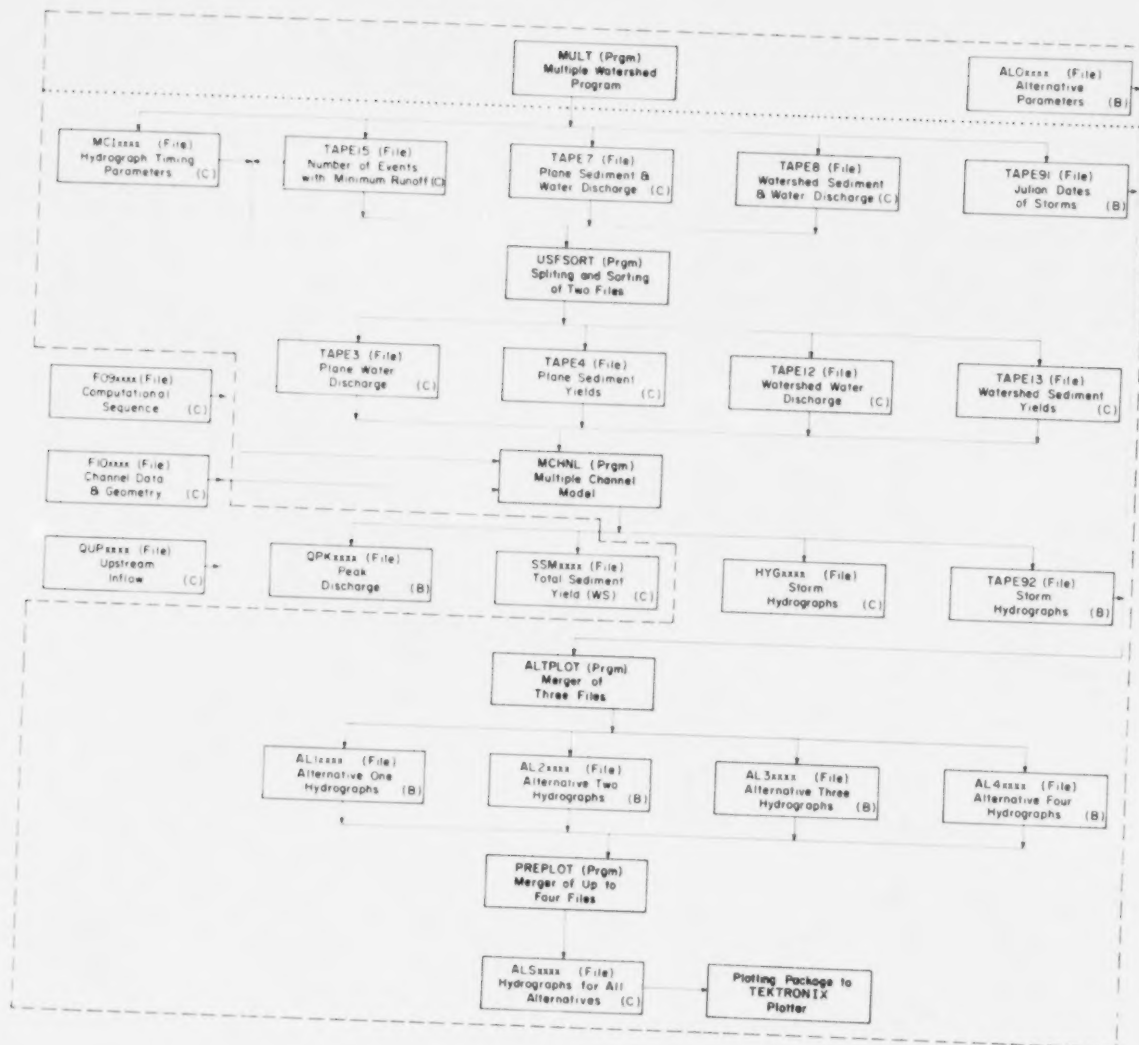


Figure 3. Water and Sediment Routing

interrelated processes involve hydrologic, biologic, atmospheric and soil-specific aspects. Therefore, a physically-based water balance model must simulate all of these aspects and properly account for their interrelation. After a literature search for an existing water balance model, a decision was made to modify and implement a simulation presented by Goldstein and Mankin (1972). This program, known as PROSPER, has been widely tested in a variety of locations with generally good results (Swift et al., 1975; Luxmoore et al., 1977). The success of these applications is probably based on the flexibility built into the model. It is possible to adapt model subroutines to a particular watershed environment by modifying the methods used to calculate the resistances to water flux through the soil and plant components.

PROSPER is a plant-atmosphere-soil water flux simulation which implements an energy balance and aerodynamic calculation of evapotranspiration using the Penman method with a multilayer Darcian soil model. The model simulates the fluxes of water through the soil and plants in response to atmospheric and solar conditions. The simulation uses a time increment of one day. All hydrologic plant and atmospheric processes are averaged on a daily basis. The model as implemented uses an electrical circuit analogue for the soil and plant system. The current in the circuit loop represents the water flux through the respective soil, plant, or atmospheric component.

Since PROSPER was written for predicting daily variations in water flux, its formulation was found to be inadequate for the prediction of infiltration and interception on a storm-by-storm basis. Further, since the time history of infiltration during a storm is particularly critical when determining water and sediment runoff, the effects of watershed management on infiltration parameters could not be properly simulated without modification to PROSPER. Therefore, the water balance component was modified to include modified Green-Ampt infiltration component for layered soils for more precise determination of storm water runoff and a storm water interception routine. Both infiltration and interception components provide greater sensitivity to management activities in the watershed than the original PROSPER, and the infiltration routine provides for the interfacing of the water balance component with the water and sediment routing and water quality subprograms.

Figure 4, which is modified from Goldstein and Mankin (1972), shows a schematic representation of the water balance system. Resistances in the current analogue are formulated in terms of physically defined watershed moisture parameters. These parameters include soil layer hydraulic conductivities, root resistance, and empirically measured species specific stomatal resistances. By varying the values of these parameters to fit the physical setting of the watershed, it is possible to achieve a more precise representation of physical processes than provided by previous simulation of similar scope.

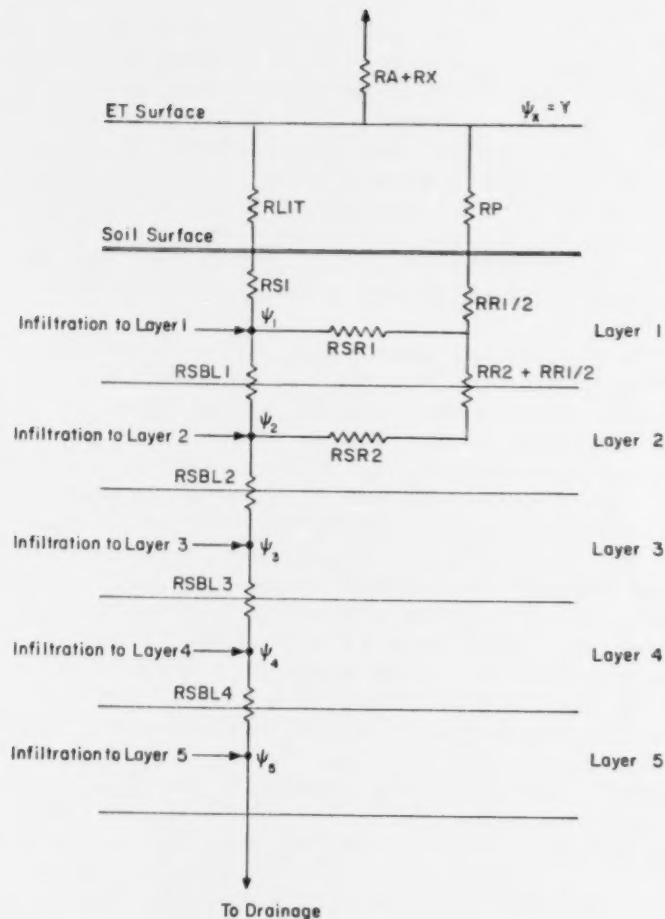


Figure 4. Soil-plant flux schematic.

In many watersheds the most significant contribution to runoff is from snowmelt. Often the most severe erosion and sedimentation events occur as combinations of high intensity spring rainfall and snowmelt runoff. This aspect of the hydrologic cycle is particularly important in high altitude watersheds of the western United States and in northern latitudes. After examining available snowmelt models, it was felt that a modified version of the model developed by Leaf and Brink (1973) would provide the best approach for incorporation into the generalized planning model. This model is a simulation originally developed for snowmelt prediction in Colorado subalpine watersheds. It is a mechanistic approach with data requirements consistent with previously described planning model components, in particular, evapotranspiration and water balance components. The snowmelt model as formulated is designed to provide daily runoff water yields and therefore, it requires modification to interface with the other water balance routines.

The snowmelt model is a combination of the fundamental laws of conservation of mass (water balance) and conservation of energy. The thermal

state of the snowpack is described by the calorie deficit of the pack on any given day. The calorie deficit is defined to be the number of calories (per unit area) required to bring the pack to an isothermal state at 0°C. It is assumed that no melt can occur until the pack has reached this state. The general scheme of computing snowpack accumulation, temperature change, and melt begins with an assessment of the effects of the day's precipitation, if any, on the pack. This includes the effect of the snow or rain introduced to the pack at the average air temperature, as well as the addition of water to the pack as snow or as rainfall. After the effect of rainfall has been assessed, an energy balance calculation is begun with a determination of the net radiation input (or loss) to the snow pack. Depending on the thickness of the snowpack and the amount of free water existing in the pack, a thermal diffusion model is used instead of a radiation balance. The new pack temperature is calculated using either the thermal diffusion model or the radiation balance directly. Based on the new temperature, a new calorie deficit or melt is calculated, and the simulation proceeds to the next day's precipitation and radiation data. Water balance relations are adjusted daily based on the above outcome.

The original Leaf and Brink (1973) snowmelt model was designed to produce daily runoff water yields. As in the previously discussed case, this type of runoff description is inadequate for prediction of the water hydrographs required for simulation of nonpoint source pollution transport. A means for transforming the melt water volume into a snowmelt hyetograph is required. Since the melt occurs in response to the input of solar energy, it seems reasonable to approximate the snowmelt hyetograph by distributing the melt over the sunshine period using the hourly insolation divided by the total daily insolation as a weighting function. Since hourly measurements of insolation at the snowpack are not usually available, it is assumed that the corresponding ratio of hourly extraterrestrial to total extraterrestrial radiation will suffice. This insolation weighted meltwater runoff function is subjected to infiltration at a rate equal to the saturated hydraulic conductivity, and the resulting excess runoff is treated by the routing models in the same manner as the excess rainfall described above.

WATER AND SEDIMENT ROUTING

The water and sediment routing model is designed to route storm water and sediment runoff from watersheds of complex geometry. In order to accomplish this task, the complex watershed geometry must be simplified into a representation suitable for computer simulation. The geometric approximation used in this component is an arbitrary number of two plane, one-channel "open book" subwatersheds and planes linked together by channels. Figure 5 illustrates a sample representation of such a system of planes and channels.

For simplicity, a numerical solution to the kinematic wave problem could have been used for both the subwatershed units and the linking channels. However, an analytical solution such as the method of characteristics approach allows more efficient use of computer storage and usually more rapid calculation of the runoff hydrograph. Therefore, whenever possible, analytical solutions are employed. To be consistent, in the portions of the watershed where the analytical methods is used to route the water, the sediment yield is also computed by an analytical method. Likewise, in the

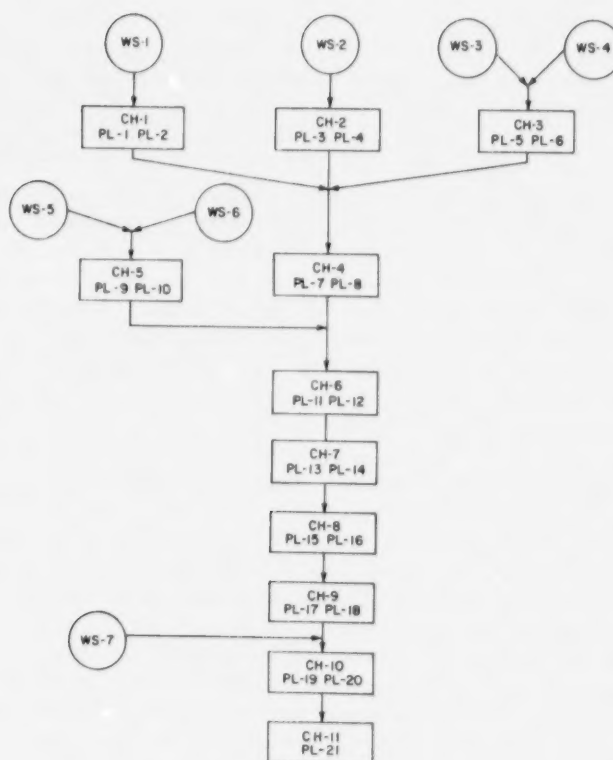


Figure 5. Schematic diagram of a typical routing network.

portions where water is routed numerically, sediment yield and transport are computed by a numerical routing scheme.

Overland Flow and Primary Channels

The two tasks performed by this program are the determination of sediment yield by size fractions and the routing of water for all plane and upstream watershed units that form the entire watershed. The water routing is performed analytically by applying the method of characteristics to the kinematic wave approximation to the momentum and continuity equations. The sediment yield is calculated by comparing the supply due to detachment by both rainfall and runoff, and the potential transporting capacity. The transport capacity is determined by size fraction. The suspended sediment transport capacity is calculated using Einstein's (1950) suspended sediment equation and bed load is calculated using the Meyer-Peter, Muller (USBR, 1960) equation. These methods are used in both the overland flow and primary channel flow routines.

Main Channel Water Routing

The main channel routing program uses a numerical scheme to route the water in the downstream main channels. It uses the discharges calculated in

the overland flow and primary channel routines as upstream and lateral inflows into the channel units. In addition, an infiltration routine calculates the amount of water which infiltrates in the channel units and subtracts it from the lateral inflow.

Sediment Routing

The sediment routing in the main channel routine uses a similar principle to the sediment yield calculations in the overland flow and primary channel model--the process of balancing supply and capacity for each sediment size. In addition, the main channel includes the effect of armoring on the sediment transport rate. The process of armoring occurs because of the difference in sediment transport capacity between the different sizes, resulting in a layer of large size fraction formed on the surface. If the erosion processes continue, this layer of larger size fraction will protect the smaller one from detaching or dislodging.

The sediment calculations in the main channel are also different in that the numerical method used allows for the sediment to be routed through the channels at each time increment and then integrated over the time increments to arrive at a total yield for each size fraction. The method used in overland flow and primary channel model can only provide a total yield for each size fraction, but cannot truly route the sediment through the channel. The main channel model is able to route the sediment because the balancing of transporting capacity and supply can be compared at each time and space increment along the channel due to the use of the numerical scheme. The method used in the overland flow and primary channel does not allow for this, since only average conditions are determined.

Since the main channel model uses the numerical method, which requires sediment transport rates at each time increment, and needs to use the yields calculated in the overland flow and primary channel model as upstream and lateral inflows, the yields in the latter must be transformed into sediment hydrographs. This is accomplished by distributing the yields from overland flow and primary channel model in proportion to the water discharge at each time increment. A more detailed discussion of the water and sediment routing component may be found in Li et al (1979).

This water and sediment routing method has been successfully employed on a variety of watersheds. Results have shown it to provide very accurate characterization of watershed hydraulics. A considerable degree of emphasis has been placed on the previously described water balance and the water and sediment routing components. Pollutants are, of course, chiefly transported by surface and subsurface flow components. Therefore, the characterization of the hydraulics of infiltration, overland flow, channel flow, and the mechanics of sediment yield and transport are critical and of central importance to accurately estimating pollutant migration. The described hydraulic components provide the level of accuracy necessary to represent these processes and through the model structure remain practical for implementation on most computing hardware.

POLLUTANT ROUTING

As originally formulated, the pollutant routing module addressed the routing of nutrients. Nutrient compounds are a source of nonpoint

pollutants affecting water quality. Nutrients, such as nitrogen and phosphorus, are of concern because of their role in eutrophication processes. A physical process simulation model was developed for predicting nutrient losses from forest and agricultural watersheds associated with surface runoff and sediment transport. Mass balance and loading function concepts were the basic principles utilized in formulating this model. The model was developed to predict loadings of organic nutrients, nitrate, ammonium, and inorganic ortho-phosphorus to streams and rivers.

Natural nutrient input to the ecosystem comes mainly from precipitation, litter fall, and geologic weathering. Precipitation and litter fall were considered the primary external inputs of nutrients from the atmosphere. These average inputs were routed into the litter layer where microbial degradation occurred. The products of degradation were then routed to the stream and into the soil layer. Within the soil layer, these products were again evaluated along with plant uptake and soil adsorption. The products of these processes occurring within the soil were then routed to the stream. Generally, nutrient constituents cannot move unless transported by sediment and water and therefore, water and sediment are the major carriers of nutrients through the ecosystem. Evaluation of these carrier amounts is necessary for predicting nutrient losses from the watershed.

The nutrient simulator proposed here is basically a nutrient budget model. All of the processes mentioned above except the immobilization process were taken into account when simulating average nutrient concentrations in the soil. The quantities of nutrient losses to streams during storms were predicted by the incorporation of the loading function concept.

As stated earlier, the pollutant model was originally developed, as described, for nutrient routing. However, it is currently under modification to allow the addition of pesticide routing processes. Again, an existing accepted pesticide model was selected. The pesticide model chosen for this study is a simple mechanistically based model developed by the Agricultural Research Service (Knisel, 1980). It is directly coupled to the water and sediment yield model through the use of water and sediment runoff volumes as inputs. The model also requires an estimate of the initial moisture deficit in the soil, which is obtained directly from the water balance routines.

The pesticide component (as reported by Knisel, 1980) was developed on simplified concepts of processes and designed to be responsive to different management options. Foliar- and soil-applied pesticides are separately described so that different decay rates can be used for each source of the same chemical if necessary. Usually pesticide residing on foliage dissipates more rapidly than that from soil. Also decay rates can be made site specific if information is available. Movement of pesticides from the soil surface as a result of infiltrating water is estimated using differences of rainfall and runoff for the storm and pesticide mobility parameters. Pesticide in runoff is then partitioned between the solution in water and that adsorbed to sediment. This aspect is particularly important when examining management options that may limit sediment yield. Further, the partition coefficient system used is particularly useful since the coefficients are lab measurable.

TEMPERATURE AND DISSOLVED OXYGEN ROUTING

Thermal energy content, dissolved oxygen (DO), and biological oxygen demand (BOD) of runoff water can directly or indirectly affect the temperature and oxygen content in the stream. Based on mass and energy balance, the temperature and dissolved oxygen model is included in this simulation. This model is useful to evaluate the thermal and dissolved oxygen loading to the stream through surface runoff.

Overland flows transmit thermal and DO loading from land surfaces to the stream. Temperature and dissolved oxygen loading of the stream result from high temperature, high bio-chemical oxygen demand, and low dissolved oxygen in runoff water. Temperature and DO effects of subsurface flow are not included in this model.

The three mechanisms of heat transfer, radiation, conduction, and convection, are included in this model. Each mechanism plays a role in the heat transfer process. Conduction is the only predominant mechanism for heat transfer between soil layers and heat transmission between soil and surface flow. Convective heat transfer occurs because of relative motion between various parts of the heated body or fluid. Convection plays an important role in heat transfer from water surfaces, particularly in evaporative processes. In this model, these heat transfer mechanisms are used to formulate equations for (1) atmospheric processes, (2) canopy-ground cover processes, and (3) surface runoff processes.

The oxygen concentration in the stream water at any given time is determined by the solubility of oxygen in the water, the rate at which this oxygen is consumed by various biological processes (represented by biological oxygen demand), and the rate at which this depletion is replenished. Deoxygenation of the water due to the bacterial decomposition of carbonaceous organic material and reaeration caused by the oxygen deficit and turbulence are the most fundamental processes occurring in natural water. The basic theory used to describe the deoxygenation and reaeration processes was proposed by Streeter and Phelps (1925). The rate at which the BOD is exerted was presumed to be identical to that observed while using the laboratory BOD test. A proportionality is assumed to exist between the reaeration rate and certain hydraulic parameters of flow. The DO effects include concentration reductions due to purging action of gases rising from the benthic layer, plant aspiration, diffusion into the benthic layer, and DO addition photosynthesis.

STREAMBANK EROSION

Much of the sediment production of watersheds and channel systems arises from streambank erosion. In the United States it has been estimated that approximately 500 million tons/year enter the drainage systems of the country (Barnes, 1963). Therefore, a mathematical model of the process of streambank erosion by channel widening is included in this simulation. The predictive capability of the model is enhanced by its phenomenological structure, although empirical data are needed in the stream morphology component. The model estimates the total amount of streambank erosion and the fraction of it that goes into suspension. Threshold channel conditions,

bank characteristics, and the hydrologic events are input to the model.

As formulated, the streambank erosion model estimates the total amount of erosion that is likely to occur in the transition from a condition of geomorphic equilibrium to another condition of equilibrium. As such, it does not provide information on the rate of streambank erosion; rather, it gives a total value assuming the new equilibrium condition is eventually reached. In practice, however, the rate of streambank erosion is a function of the time history of hydrologic events which are not explicitly considered in the present model. Therefore, the calculated values are to be regarded as estimates of the total amount of streambank erosion that is associated with a certain level of hydrologic excitation. Further refinements will need to be implemented if the model is to provide information on the rate of streambank erosion.

FOREST LITTER

A first approximation to the rill formation and the loading of forest litter is included in this simulation. The model is based on the assumption that the amount of forest litter loading is directly proportional to the areal extent of rilling. This is a reasonable assumption in view of the demonstrated effectiveness of concentrated flow in transporting sediment and debris through upland watershed drainage networks. This approach allows the conversion of the forest litter loading problem into that of determining the areal extent of rilling (rilling density), given a set of topographic, hydrologic, and morphologic conditions.

The quantity of forest litter delivered to a stream is a direct function of the areal extent of rilling and the amount of forest litter production. The areal extent of rilling will, in general, be determined by large events. Smaller subsequent events will not entirely fill the established rill network. Therefore, the litter washed out of the rill network will be detached from the area defined by the top width of the flowing water. This top width may be obtained from an "at a station" relationship provided by Li, Simons and Stevens (1976).

SEQUENTIAL GOAL PROGRAMMING

The problem of management of a basin depends very much upon the selection of the best land-use strategy to optimize specified socioeconomic objectives under certain constraints on water and sediment transport conditions in the stream channels and on water quality standards in the basin. The quality of a basin management plan depends on the quality and availability of data. However, the efficiency of planning depends on the optimization tool to be selected and the accuracy of simulation models used to project future basin-system response. A review and evaluation of multi-objective programming techniques used to solve basin planning problems has been conducted by Loucks (1975) and later by Cohon and Marks (1975). One technique of increasing popularity often encountered in water resources literature is goal programming. It is designed to evaluate (possibly conflicting) goals as well as goals of differing priorities.

The planning model is designed as multi-level. The lower level is used to select optimal land-use strategies based on different alternative

management practices and subject to some specified land-use constraints. In this level, the elements of the resource response matrix are calculated by various land-use process models which use the same type of spatial and temporal information (i.e., same soil-vegetation units and time-step). After selecting a set of optimal strategies for land-use management, the upper level is then used to select the best management strategy for the entire basin system based on different land-use strategies selected from the first level and the outcomes of hydrologic and water quality component processes, subject to some specified socioeconomic constraints. In this level, complex models of water and sediment yields and water quality are used to calculate the elements of the process response matrix which then serves as the necessary input for the selection of the best basin management strategy.

POTENTIAL APPLICATIONS

Potential applications of the simulation are initially intended to aid in the evaluation of watershed management alternatives. Management activities that could be considered initially were vegetation growth (over story and under story), timber harvest, foliage utilization (grazing), site preparation, waste disposal and prescribed nutrient and pesticide applications. The linkage of the management activity models, the process models and the multiple objective programming model form the preliminary generalized planning model. This preliminary planning model would be useful in evaluating selected alternatives as a function of environmental goals. Environmental goals relate to control of sediment, nutrients, pesticide, hazardous waste, thermal, and dissolved oxygen pollution. Also, resource management goals can be identified that maximize income and/or implement the best management practices. The changes caused by management activities are reflected in changes in input model parameters. The effects of these changes may be simulated by adjusting soil and vegetative and chemical parameters. Chronological simulation allows those parameters to be varied either instantaneously or as functions of time. By interfacing with the planning model, these parameter values may be adjusted in order to select the best management alternatives and scheduling.

ACKNOWLEDGEMENT

The writers wish to acknowledge Mr. W. T. Fullerton, Mr. J. N-H. Ho, Dr. N. Duong, and Dr. V. M. Ponce for their valuable contributions to the formulation of this simulation. Financial support for this study was provided by the Environmental Protection Agency, Environmental Research Laboratory, Athens, Georgia; and by the USDA Forest Service, Rocky Mountain Forest and Range Experiment Station, Flagstaff, Arizona.

REFERENCES

- Barnes, R. C., Jr. 1968. Streambank erosion. *Soil Conservation* 33(6):126-128.
- Cohon, J. L., and D. H. Marks. 1975. A Review and Evaluation of Multiobjective Programming Techniques. *Water Resource Research* 11(2):208-220.

- Einstein, A. H. 1950. The Bed Load Function for Sediment Transportation in Open Channel Flows. U.S. Department of Agriculture Technical Bulletin No. 1026.
- Goldstein, R. A., and J. B. Mankin. 1972. PROSPER: A Model of Atmosphere-Soil-Plant Water Flow. Proceedings, Summer Simulation Conference, Los Angeles, pp. 1176-1181.
- Knisel, W. G. 1980. "A Field Scale Model for Chemicals, Runoff, and Erosion from Agricultural Management Systems," for U.S. Department of Agriculture, Conservation Research Report #26.
- Leaf, C. F. and G. E. Brink. 1973. Computer Simulation of Snowmelt Within a Colorado Subalpine Watershed, USDA Forest Service Research Paper RM-99, Rocky Mountain Forest and Range Experiment Station, Fort Collins, Colorado.
- Li, R. M., D. B. Simons, W. T. Fullerton, K. G. Eggert and B. E. Spronk. 1979. "Simulation of Water Runoff and Sediment Yield from a System of Multipole Watersheds," presented at the XVIII Congress of the International Association for Hydraulic Research, Cagliari, Italy.
- Li, R. M., D. B. Simons, and M. A. Stevens. 1976. Morphology of Cobble Streams in Small Watersheds. J. Hydraul. Div., ASCE, 12(HY8):1101-1112.
- Loucks, D. P. 1975. Conflict and Choice: Planning for Multiple Objectives. In: Economy Wide Models and Development Planning, edited by C. Blitzler, P. Clark, and L. Taylor, Oxford University Press, New York.
- Luxmoore, R. J., D. J. Van Roagen, F. D. Hale, J. B. Mankin, and R. A. Goldstein. 1977. Field Water Balance and Simulated Water Relations of Prairie and Oak-Hickory Vegetation on Deciduous Forest Soils. Soil Sci. 123(2):77.
- Streeter, H. W., and E. B. Phelps. 1925. A Study of the Pollution and Natural Purification of the Ohio River. Public Health Bulletin 146, U.S. Public Health Service, Washington, D.C.
- Swift, L. W., W. T. Swank, J. B. Mankin, R. J. Luxmoore, and R. A. Goldstein. 1975. Simulation of Evapotranspiration and Drainage from Mature and Clear-Cut Deciduous Forests and Young Pine Plantations. Water Resources Research 11(5):667.
- U.S. Bureau of Reclamation. 1960. Investigation of Meyer-Peter, Muller Bedload Formulas. Sedimentation Section, Hydrology Branch, Div. of Project Investigation, U.S. Department of the Interior.

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

DATA MANAGEMENT FOR CONTINUOUS

HYDROLOGIC SIMULATION

Jy S. Wu, Ph.D.¹

INTRODUCTION

The use of models to simulate rainfall-runoff events is receiving considerable attention by water resources planners and engineers. One of the major reasons for this concern is the limited capability of field sampling programs in obtaining a continuous data-base which reflects the watershed responses to changes in hydrologic conditions and management practices over an extended period of time. The conduct of a continual field monitoring and sampling program is often quite expensive; even a well managed sampling program can still be handicapped by the failure of sampling devices and unexpected difficulties during the sampling period. Mathematical modeling thus becomes a useful tool for analyzing existing data and predicting future conditions as a result of implementing watershed management practices. In some cases, the collected data must be utilized to statistically generate missing records for calibrating a continuous hydrologic simulation model. This paper describes the application of the U.S. EPA's Agricultural Runoff Model (ARM) on one of the agricultural watersheds of the Chowan River Basin in North Carolina. The technique of regression analysis was employed to generate missing runoff data as required for calibrating the ARM model.

WATERSHED DESCRIPTION

The Cutawhiskie Lateral monitoring site is located in Hertford County of northeastern North Carolina with a total drainage area of 667 acres. Runoff from this monitoring site drains into the Cutawhiskie Creek which, in turn, drains into the Chowan River near the North Carolina-Virginia border. The site is relatively flat with an average overland slope of 0.4%. Land use is about 50% woodland and 50% cropland of peanut, soybean, cotton and corn.

The North Carolina State University, in cooperation with the Division of Environmental Management, has been conducting a field sampling and monitoring program for selected agricultural watersheds including the Cutawhiskie site, to assess nonpoint source effects on streams. The program also calls for the survey of producers within the monitored watersheds to assess land use, management practices and opinions on 208 planning and

¹ Assistant Professor, Department of Urban and Environmental Engineering, University of North Carolina at Charlotte, Charlotte, N.C. 28223

implementation options. A field sampling data-base and survey information have been collected for a period of three years.

RESEARCH RATIONALE

The selected period for hydrologic calibration of the ARM model was chosen from October, 1980, through September, 1981. The hourly rainfall and daily evaporation records were incomplete during this particular period. It was necessary to explore alternate procedures for generating missing records. Available data from nearby meteorological stations were utilized to construct isohyets of rainfall and evaporation. Estimations were then based on interpolating adjacent isohyets of the study area.

The recorded runoff information was also incomplete. A total of eleven runoff events having well defined hydrographs were identified. The technique of multi-regression analysis was employed to generate missing runoff records. Runoff volume of each event was correlated with rainfall volume of each event, duration of rainfall, and number of dry days prior to rainfall. A regression equation was developed and used to calculate the single event, monthly and annual runoff volumes. This allows calibrating the ARM model based on annual and monthly water balances and individual storm comparison. A detailed description of data preparation is given in subsequent sections.

ARM MODEL DESCRIPTION

The ARM model is a continuous hydrologic and water quality simulation model (Donigian & Harley, 1978). It simulates runoff, sediment, pesticides and nutrients contribution to stream channels from both surface and sub-surface sources. Major data requirements for hydrologic simulation include:

- A. Rainfall records on 5 minutes, 15 minutes, or hourly intervals.
- B. Daily evaporation data, and the following hydrologic parameters:

- Nominal upper zone soil moisture storage, UZSN
- Initial upper zone soil moisture storage, UZS
- Nominal lower zone soil moisture storage, LZSN
- Initial lower zone soil moisture storage, LZN
- Overland flow length, L
- Average overland flow slope, SS
- Fraction of impervious area, A
- Maximum interception storage, EPXM
- Index to actual monthly evaporation, K3
- Potential evapotranspiration correction factor, PETMUL
- Mean infiltration rate, INFIL
- Interflow parameter, INTER
- Interflow recession rate, IRC
- Fraction of groundwater recharge percolating to deep groundwater, K24L
- Groundwater recession rate, KK24
- Fraction of watershed area where groundwater is within reach of vegetation, K24EL

Initial groundwater storage, SWG
Parameter for variable recession rate of groundwater
recharge, KV
Initial interception storage, ICS
Initial overland flow storage, OFS
Initial interflow storage, IFS

Of all the above parameters; LZSN, INFIL, UZSN, INTER, and K3 are the major parameters subject to calibration. Hydrologic calibration involves the comparison of simulated and recorded runoff volumes on annual, monthly and single storm event basis.

INPUT DATA

Rainfall

Rainfall records were obtained from the U. S. Geological Survey (U.S.G.S.) meteorological station no. 02053175 at Cutawhiskie Creek tributary near Menola, N.C. The recorded rainfall amounts were found to fluctuate due to the expansion and contraction effect of the rainwater collecting devices. It was necessary to compare rainfall records with the nearby Murfreesboro and Lewiston stations to ensure the probable occurrence of rainfall. If no rain was noted from these two reference stations, the recorded fluctuations of rainfall amounts were ignored for the Cutawhiskie site. The U.S.G.S. data provides rainfall records from October through December of 1980; and from January, April, May, June and July of 1981. The missing rainfall records for February, March, August and September of 1981 were estimated by constructing rainfall isohyets from recorded data of the nearby stations. Once the monthly rainfall for those missing months were estimated, the hourly rainfall records from the Murfreesboro station were adjusted proportionally and used as the input hourly rainfall for the Cutawhiskie site. The monthly rainfall data are given in Table 1.

Evapotranspiration

Potential evapotranspiration is assumed to be equal to lake evaporation estimated from Weather Bureau Class A pan records. The ARM model allows the use of a single variable to adjust pan evaporation data. The actual evapotranspiration is computed as a function of the potential evapotranspiration and soil moisture conditions. Chang (1968) reported average ratios between potential evapotranspiration and pan evaporation. These ratios for cotton, grass and corn are summarized in Table 2.

Table 1.
Rainfall Data for Cutawhiskie Site, 1980-1981

Month	Rainfall, in			
	U.S.G.S.*	Cutawhiskie	Murfreesboro	Lewiston
October	6.23	4.78	7.02	3.96
November	3.42	2.66	2.02	2.13
December	4.59	2.13	2.13	1.71
January	4.64	1.02	0.94	1.19
February	--	2.55**	2.65	2.25
March	--	2.20**	2.42	2.03
April	5.00	2.00	2.29	1.58
May	4.88	3.55	3.16	4.11
June	4.24	3.21	4.35	3.36
July	4.84	3.81	2.29	2.98
August	--	7.00**	7.10	6.86
September	--	1.20**	1.20	--
		36.11	37.57	

* Values include effects of equipment expansion and contraction and may be high.

** Estimated from isohyets.

Table 2.
Ratio of Potential Evapotranspiration to Pan Evaporation
(After Chang, 1968)

Types of Crop	Ratio		
	Young	Mature	Ripening
Cotton	0.2	0.85-1.00	0.1-0.4
Corn	0.27	0.90	0.4
Grass	0.84	(average)	

There are four meteorological stations recording pan evaporation data near the study area, namely John H. Kerr Dam in Virginia; Chapel Hill, Aurora and Hoffman Forest (formerly Maysville) in North Carolina. The following guidelines were adapted to estimate the evaporation records for the study area.

1. John H. Kerr Dam is the closest station; therefore, the evaporation data was taken directly from this station except the missing period for November and December of 1980 and January through March of 1981.
2. During the missing period, evaporation data from Chapel Hill or Hoffman Forest was used as the substitution. Preference is given to Chapel Hill since it is closer to the study area.

Table 3 summarizes the monthly evaporation data estimated for the Cutawhiskie site.

Table 3.
Monthly Evaporation Data, 1980-1981

Month	Monthly Evaporation, in			
	Cutawhiskie (Estimated)	John H. Kerr Dam	Chapel Hill	Hoffman Forest
October	3.39	3.39	3.27	3.37
November	2.89	--	2.89	1.97
December	1.54(1)	--	--	--
January	1.86(2)	--	--	--
February	2.24(3)	--	--	--
March	3.98	--	--	3.98
April	5.16	5.16	5.86	6.21
May	4.98	4.98	6.69	7.14
June	7.23	7.23	7.58	7.22
July	6.60	6.60	7.51	6.79
August	5.45	5.45	6.08	4.76
September	4.80	4.80	4.48	4.86
	50.12			

- 1) Based on Hoffman Forest data from December 7-20 and average value from December 7-20 for the remaining days.
- 2) Based on Hoffman Forest data of 1980.
- 3) Based on the partial available data from Hoffman Forest and average value for the remaining days.

Parameter Evaluation

The following summarizes calibrated values of the ARM model parameters for hydrologic simulation (see previous section for parameter definitions).

UZSN: 0.93 inch	UZS: 0.72 inch
LZSN: 6.60 inch	LZS: 6.52 inch
L: 800.0 ft.	SS: 0.40%
A: 0.00%	EPXM: 0.15 inch
PETMUL: 0.50	INFIL: 0.05 in/hr
INTER: 2.00	IRC: 0.50
K24L: 1.00	
K3: 0.5,0.5,0.5,0.6,0.67,0.7,0.75,0.8,0.8,0.8,0.65,0.5	
KK24,K24EI,SWG,KV,ICC,OFS,IFS: 0.00	

The nominal capacity of soil was estimated using the information provided by the ARM model user's manual and was subsequently fine-tuned in the calibration process. The pan evaporation data was adjusted by a factor of 0.5 in accordance with Chang's work (Chang, 1968) as described in the previous section. It was found during trial runs that values of greater than 0.5 would cause a large loss of water due to evapotranspiration, making annual water balance undesirable. The infiltration capacity, INFIL, was estimated from soil characteristics and was fine-tuned during calibration. The INTER parameter which controls runoff from interflow and peaking of runoff hydrographs was initially estimated from guidelines given in the user's manual and was subject to calibration. The K3 parameter was based on 50% watershed area with deep vegetation and was varied monthly to reflect seasonal cropping periods. Other parameters were estimated following the guidelines given in the user's manual.

RUNOFF DATA

Calibration involves the comparison of simulated and recorded runoff volumes. The existence of a good data-base of runoff records is essential for model calibration. Discharges from the Cutawhiskie site were recorded by U.S.G.S. The flow records were incomplete for five months and totally missing for three months. Therefore, it was necessary to generate flow records using the technique of multi-regression analysis. A similar analysis was used by the Principal Investigator (Wu and Ahlert, 1978) in a study to predict total runoff and sediment loads during selected runoff events.

In this project, runoff volume of each event was correlated with rainfall volume of each event, number of dry days prior to rainfall, and duration of rainfall. A total of fifteen runoff hydrographs were obtained; however, only eleven of them were used in the regression analysis. This is because the other four hydrographs were recorded during the period in which rainfall data was estimated from nearby stations. There is no assurance as

to the consistence of the timing of rainfall-runoff events. The following procedures were used for generating runoff data:

1. Hydrographs were plotted from U.S.G.S. data.
2. Area under the hydrograph was computed to derive the volume of direct runoff for each event.
3. Number of dry days prior to rainfall and duration of rainfall were determined from the continuous hourly rainfall records. A minimum of 3 hours was chosen to define an individual rainfall event.
4. Regression analysis were performed using the Minitab Computer Program (Ryan et al., 1976) available at the computer center of the University of North Carolina at Charlotte. Table 4 summarizes runoff data used for regression analysis. The standard residuals of prediction were also included in Table 4. Standard residuals normally range from -2 to +2 for good correlation. Figs. 1-4 illustrate relationships among recorded runoff, rainfall, dry days, rainfall duration and results of regression analysis.

Table 4.
Rainfall-Runoff Events for Regression Analysis

Date	Recorded Runoff (R), in	Rainfall (P), in	Dry Days (D), days	Rainfall Duration (RD), hrs	Standard Residuals From Regression
10/19/80	0.043	1.32	7.21	13	-1.64
10/25/80	0.298	1.93	5.04	14	1.01
10/30/80	0.031	0.52	1.33	17	0.56
11/21/80	0.017	0.54	2.83	8	-0.45
11/24/80	0.151	1.06	2.08	6	0.07
12/10/80	0.055	0.99	12.42	14	-0.21
1/21/81	0.045	0.82	13.83	16	0.39
4/05/81	0.225	0.74	6.21	5	2.13
4/19/81	0.029	0.66	10.63	4	-0.55
4/24/81	0.010	0.30	3.63	2	-0.29
7/03/81	0.056	0.73	0.79	3	-0.79

A regression equation was developed:

$$R = 0.1676(P) - 0.0031(D) - 0.0043(RD)$$

This equation was then applied to each rainfall event to calculate the per storm runoff volume for periods in which missing records were noted. The monthly and annual runoff volumes were computed as the summation of individual storm runoff volume.

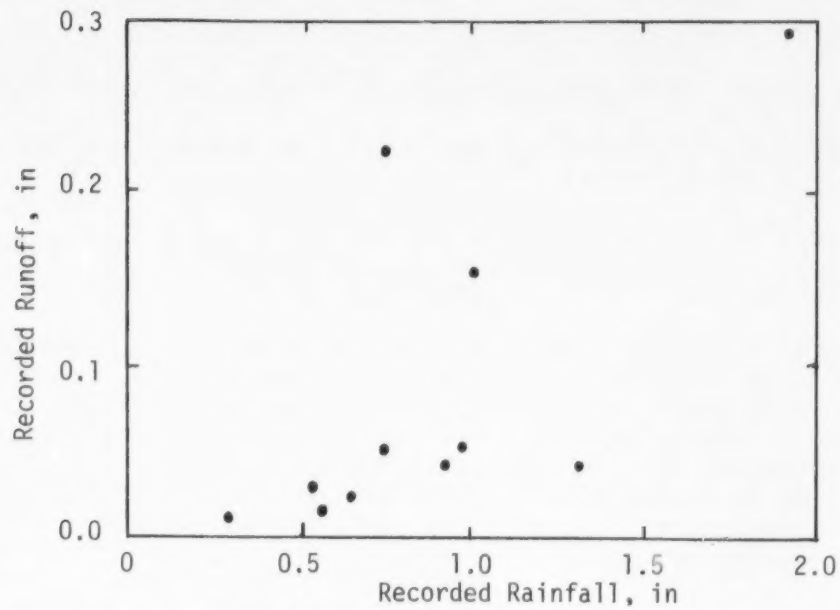


Fig.1 Runoff versus Rainfall for Storm Events Shown in Table 4

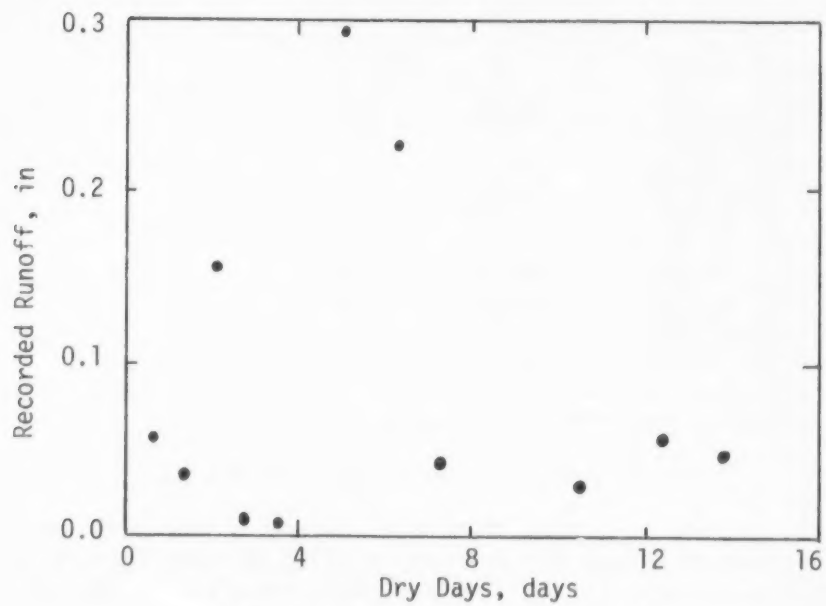


Fig.2 Runoff versus Dry Days for Storm Events Shown in Table 4

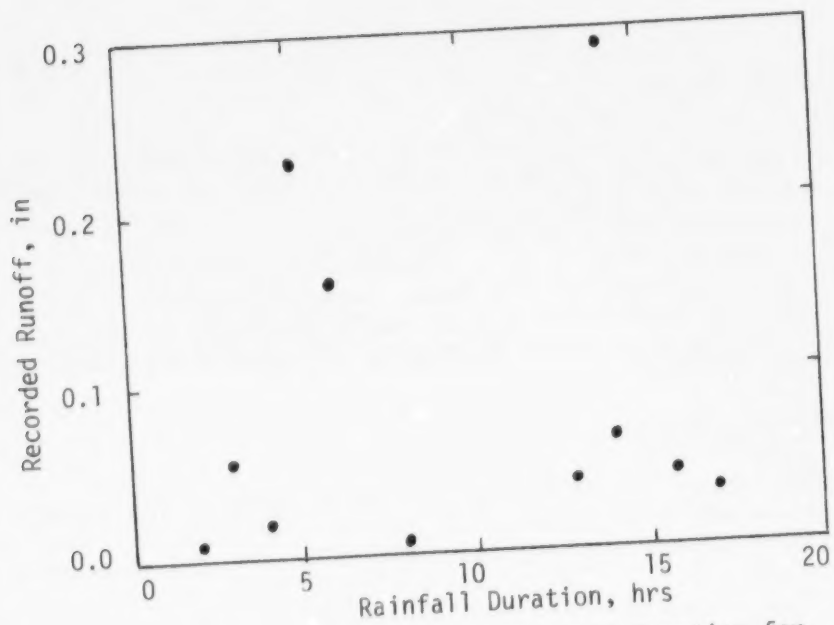


Fig.3 Runoff versus Rainfall Duration for Storm Events Shown in Table 4

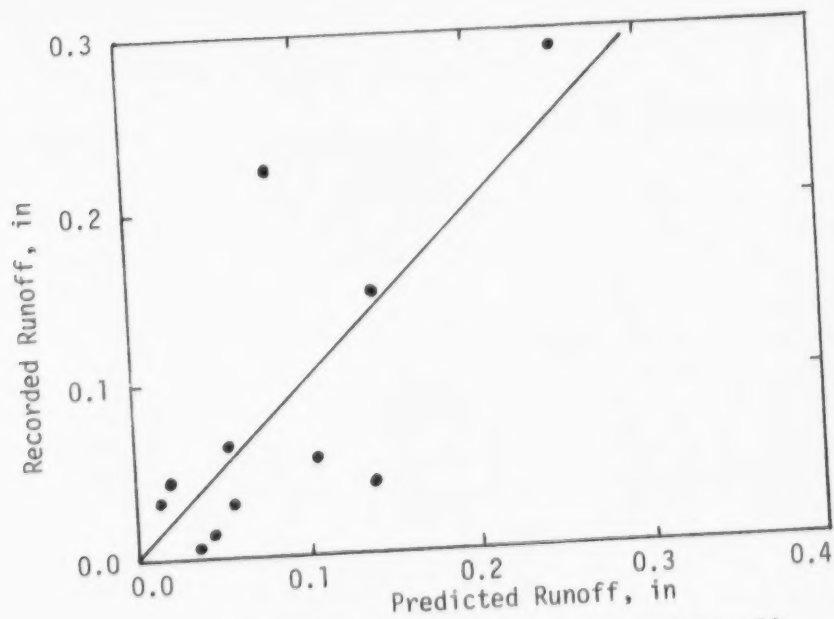


Fig.4 Recorded versus Predicted Runoff from Regression Analysis

RESULTS AND DISCUSSION

Statistical Analysis

For regression analysis, a good size of sample population is needed to obtain statistically reliable results. A total of eleven runoff events were employed in the regression analysis; a majority of them occurred in the Fall and Spring with only one runoff event recorded in July. Rainfall-runoff relationships usually vary seasonally; therefore, the regression results of runoff volume for the Summer season are unlikely to be accurate. However, due to the fact that a good runoff data-base is unavailable, the regression technique may serve as a guide for generating missing records to calibrate the ARM model.

Results of Calibration

A comparison of runoff volumes between the field data and the ARM model output is given in Table 5. It can be seen that the annual and monthly runoff volumes compare reasonably well. Figure 5 presents the model output of monthly rainfall, runoff, evapotranspiration, soil moisture changes, and groundwater recharge. The per storm comparisons are given in Figures 6-11 and in Table 6; they also agree reasonably with recorded hydrographs. Thus, the ARM model has been successfully calibrated for the Catawhiskie site. The calibration of water quality parameters could not be undertaken since there is only one set of water quality data available for the study period.

Table 5.

Predicted Runoff for the Catawhiskie Site, 1980-1981

Month	ARM Output	Runoff, in	
		U.S.G.S. Data + Regression	U.S.G.S. Records
October	0.379	0.377	0.372
November	0.210	0.153	0.153
December	0.166	0.076	0.035*
January	0.053	0.035	0.034*
February	0.377	0.552	0.552
March	0.108	0.146	0.146*
April	0.066	0.160	0.160
May	0.090	0.299	**
June	0.599	0.414	**
July	0.126	0.473	0.050*
August	1.123	0.903	0.059*
September	0.049	0.107	**
	3.346	3.695	

* Partial Records

** Missing Records

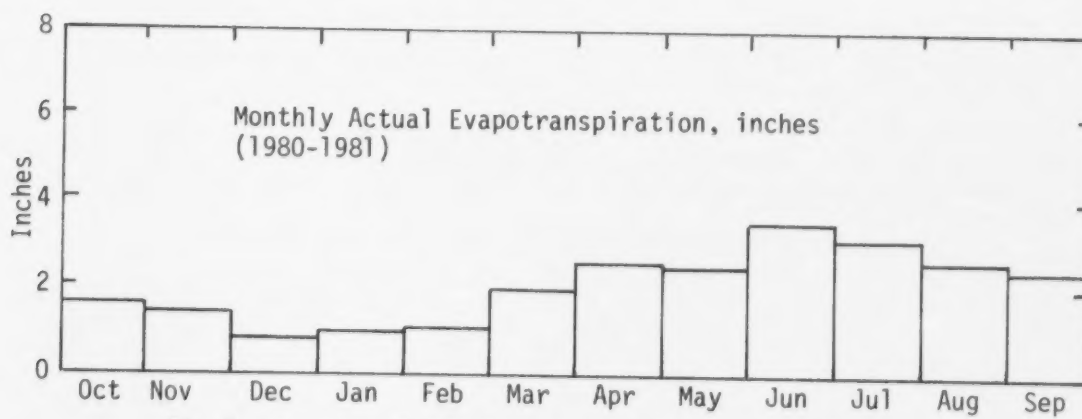
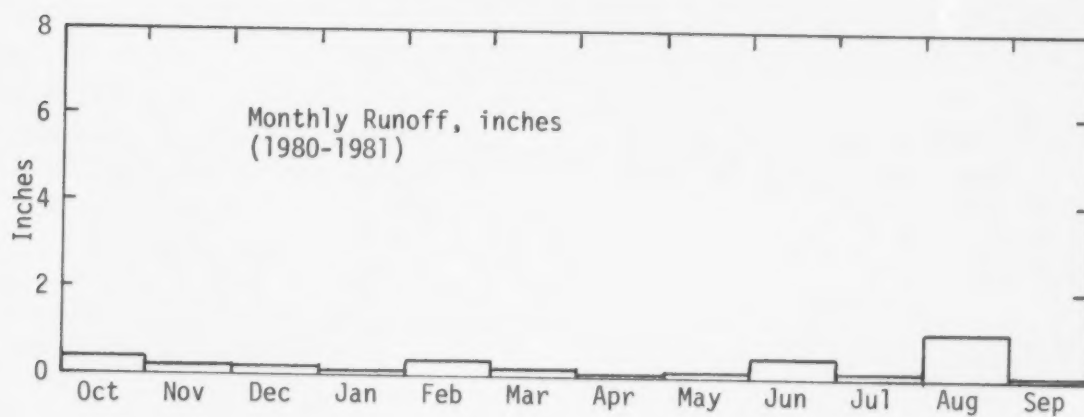
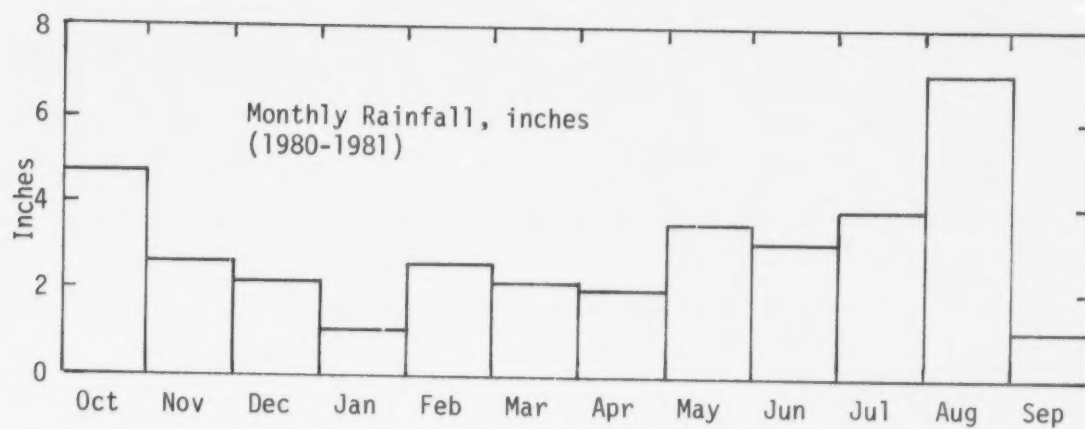


Fig.5 Summary of Simulation Results from ARM Model

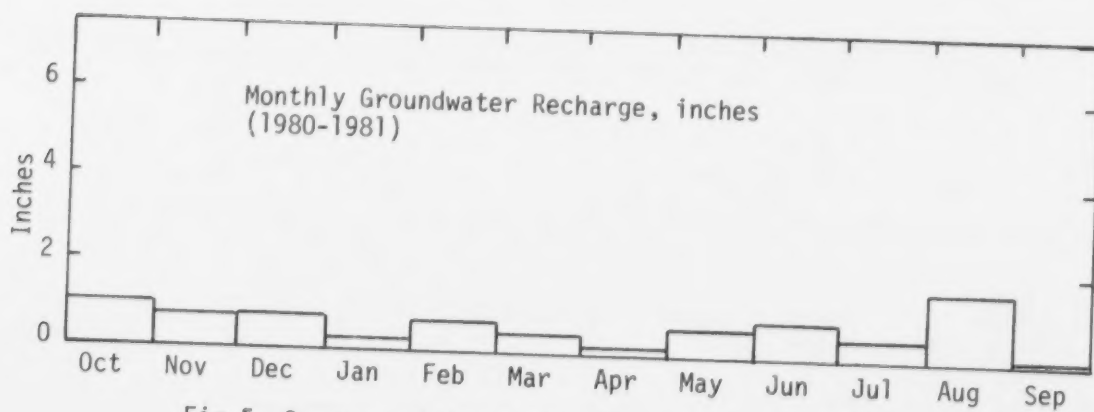
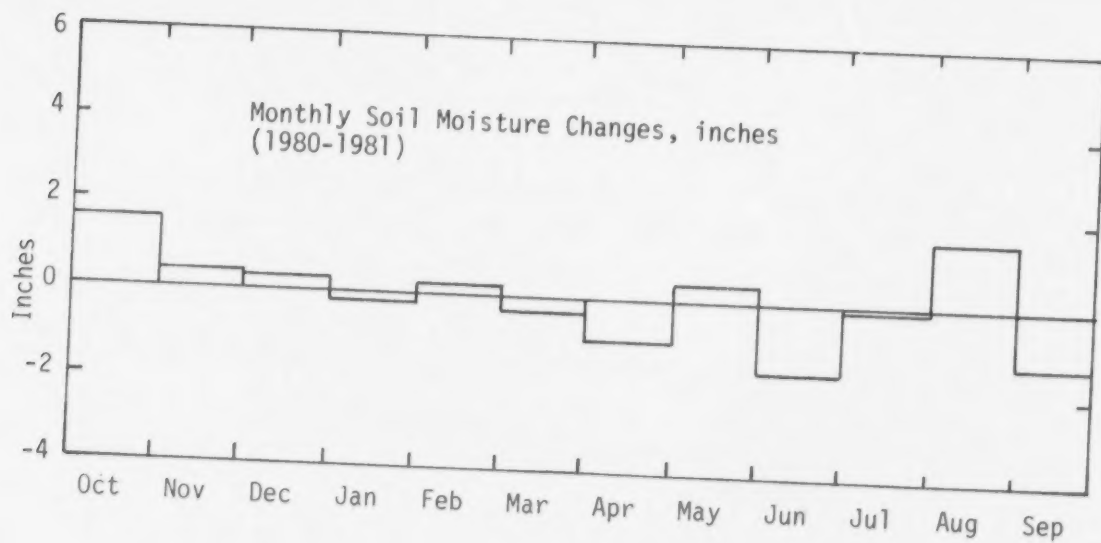


Fig.5 Summary of Simulation Results from ARM Model
(Cont'd)

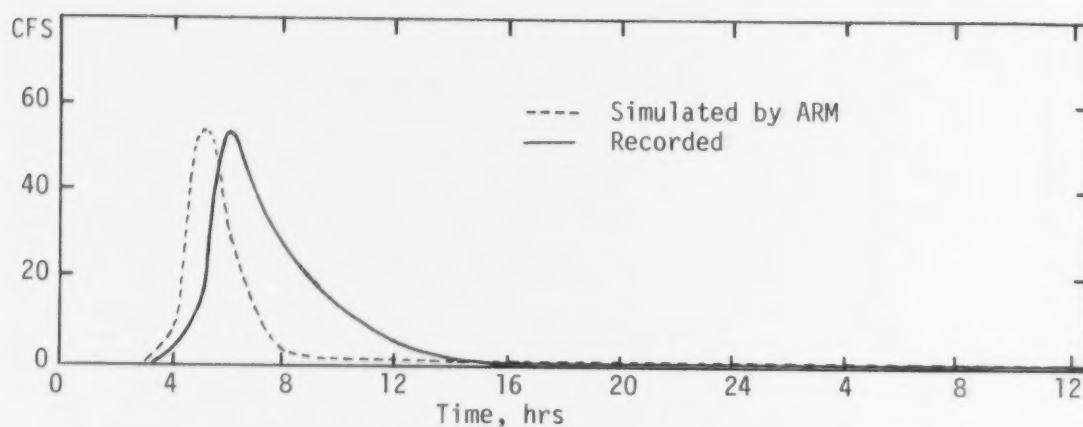


Fig.6 Hydrograph Comparison for Storm 10/25/80

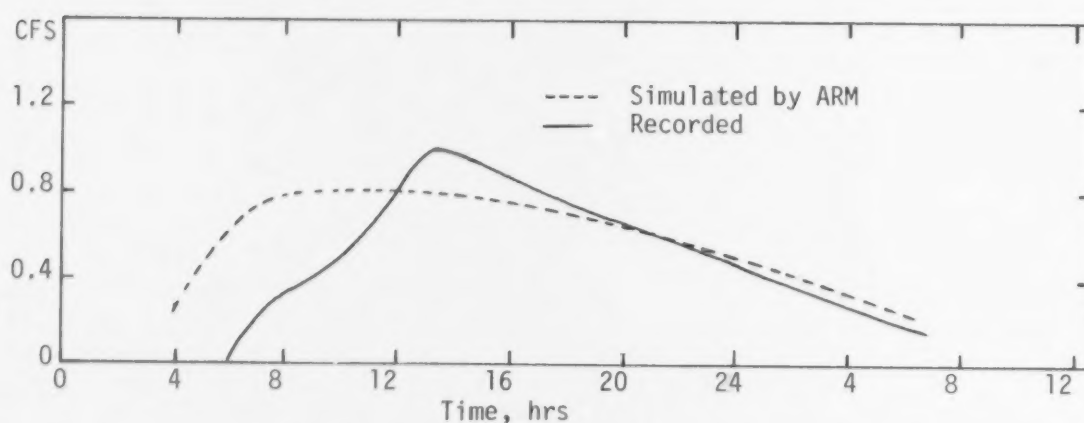


Fig.7 Hydrograph Comparison for Storm 10/30/80

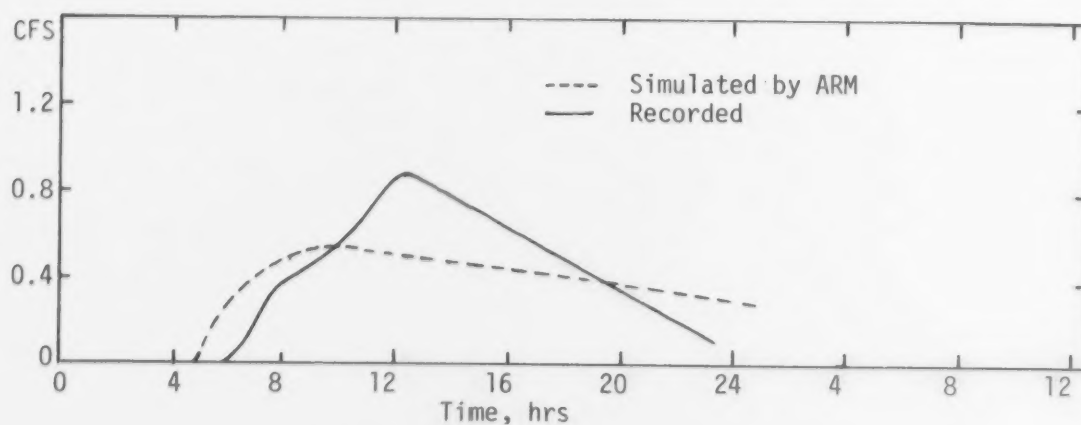


Fig.8 Hydrograph Comparison for Storm 11/21/80

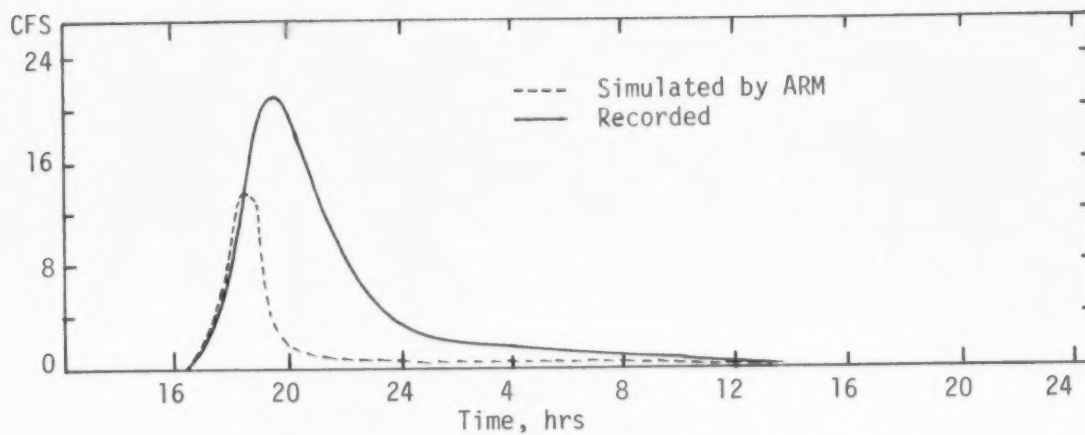


Fig.9 Hydrograph Comparison for Storm 11/24/80

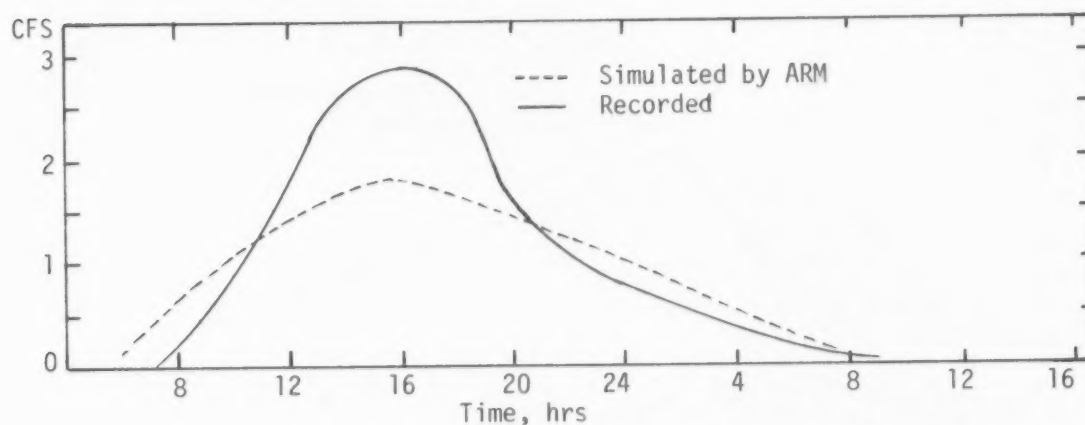


Fig.10 Hydrograph Comparison for Storm 12/10/80

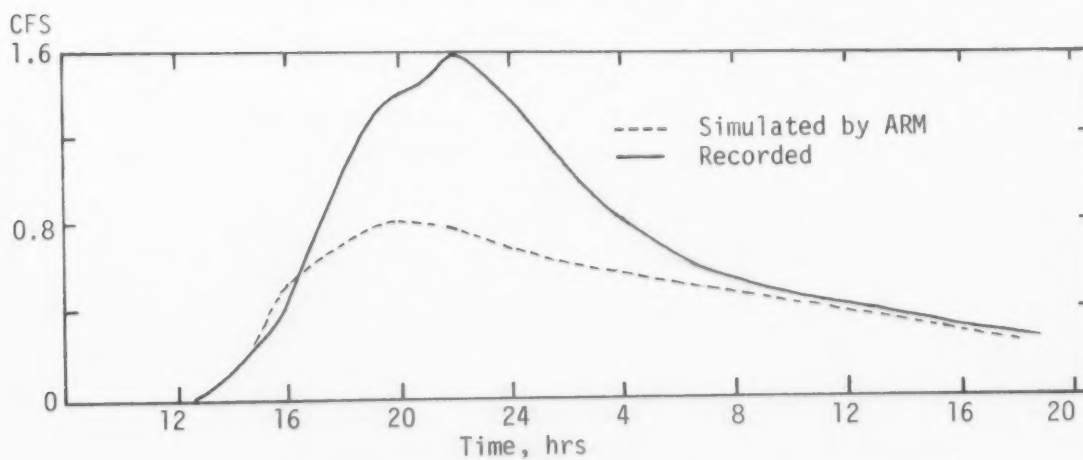


Fig.11 Hydrograph Comparison for Storm 1/21/81

Table 6.
Comparison of Single Runoff Event
from

Recorded Data and ARM Model Output

Date	Peak Time		Peak Discharge, CFS	
	Recorded	Computed	Recorded	Computed
10/25/80	6:30	5:00		
10/30/80	13:30	12:00	53.0	54.5
11/21/80	13:00	10:30	1.0	0.8
11/24/80	19:20	18:20	0.9	0.5
12/10/80	15:30	15:00	20.8	14.2
1/21/81	22:00	19:00	2.8	1.9
			1.6	0.9

CONCLUSIONS

1. Use of continuous hydrologic simulation model can be a useful tool for evaluating the effectiveness of watershed management practices over an extended period of time. However, it requires the existence of a good data-base for calibration and verification.
2. The regression technique can be used to generate missing records for calibration; however, great care is needed in interpreting the regression results.
3. A well planned sampling program is critical so that the collected data can be utilized and served for its intended purpose of the sampling and analysis program.
4. As there is a few evaporation measuring stations in the Chowan Basin, a study is recommended to better determine the evapotranspiration of the Chowan Basin.

ACKNOWLEDGEMENT

This study was supported by the North Carolina Water Resources Research Institute. Appreciation is due Ms. Beverly Young for providing field data and other pertinent information, and Mr. Robert Davis for preparing input data and performing computer work.

REFERENCES

- Chang, Jen-Hu, "Climate and Agriculture-An Ecological Survey", Aldine Publishing Company, 1968.
- Donigian Jr., A. S. and Harley H. Davis, Jr., "User's Manual for Agricultural Runoff Management Model", EPA 600/3-78-080, 1978.
- Ryan Jr., T. A., et al. "Minitab Student Handbook", Belmont, California, 1976.
- Wu, Jy S. and Robert C. Ahlert, "Assessment of Methods for Computing Storm Runoff Loads", Water Resources Bulletin, Vol. 4, No. 2, pp. 429-439, 1978.

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

ESTIMATION OF MISSING VALUES IN MONTHLY RAINFALL SERIES

by

Efi Foufoula-Georgiou¹

ABSTRACT

Infilling of missing values is often necessary prior to the practical use of hydrological time series. In this paper, three different types of infilling methods are considered reflecting the following basic ideas:

- (1) the use of regional-statistical information in four simple techniques:
 - mean value method (MV),
 - reciprocal distance method (RD),
 - normal ratio method (NR),
 - modified weighted average method (MWA);
- (2) the use of a univariate stochastic (ARMA) model which describes the time correlation of the series;
- (3) the use of a multivariate stochastic (ARMA) model which describes the time and space correlation of the series.

An algorithm for the recursive estimation of the missing values by a parallel updating of the univariate or multivariate ARMA model is proposed and demonstrated. All methods are illustrated in a case study using 55 years of monthly rainfall data from four south Florida stations.

INTRODUCTION

Many different kinds of statistical analyses may be performed on a given data set, e.g., determination of elementary statistical parameters, auto- and cross-correlation analysis, spectral analysis, frequency analysis, fitting time series models. For routine statistics (e.g., calculation of mean, variance and skewness) missing values are seldom a problem. But for techniques as common as autocorrelation and spectral analysis missing values can cause difficulties. In multivariate analysis missing values result in "wasted information" when only the overlapping period of the time series is used in the analysis, and in numerical inconsistencies (Valencia and Schaake, 1973; Fiering, 1968; Slack, 1973) when the incomplete series are used. The evaluation of the estimation methods analyzed has utilized monthly rainfall records from the South Florida Water Management District (SFWMD), and has been based upon: a) the statistical comparison of the methods to each other at a fixed level of percent of missing values, and b) the performance of each individual method at different levels of percent of missing values. Gaps (missing values) have been artificially created in the complete record of the interpolation station (station whose missing values are to be estimated) with

¹Graduate student, Department of Environmental Engineering Sciences, University of Florida, Gainesville.

the following procedure: First, the lengths of the gaps have been generated from a discrete exponential distribution with mean $\bar{k}$ months. Then, for a given percent of missing values, m , the mean interevent length (missing values between two successive gaps), $\bar{\tau}$, has been calculated as $\bar{\tau} = \bar{k} (100-m)/m$ and the interevent lengths have been generated randomly from an exponential distribution with mean $\bar{\tau}$. The values used for $\bar{k}$ and m are based on a frequency analysis of missing values in SFWMD monthly rainfall records (Foufoula-Georgiou, 1982) and are: $\bar{k} = 2.4$ months and $m = 2, 5, 10, 15$ and 20% . Overlapping and concurrent periods of 55 years of monthly rainfall data of the four SFWMD stations shown in Fig. 1 have been used in the analysis.

TRADITIONAL ESTIMATION TECHNIQUES

Mathematical Representation

In all the following equations y will be used for the interpolation station and x_j for index station j , $j = 1, 2, \dots, n$. An estimated value at time t is y'_t .

Mean Value Method (MV) --

The simplest method simply replaces the missing values with the sample mean:

$$y'_t = \bar{y} \quad (1)$$

This method results in a reduced variance and a spurious correlation coefficient especially at a high percent of missing values.

Reciprocal Distance Method (RD)

A missing value y_t is estimated as:

$$y'_t = \sum_{j=1}^n a_j x_{j,t} \quad (2)$$

The weighting coefficients a_j are calculated from:

$$a_j = (1/d_j)^p / \sum_{j=1}^n (1/d_j)^p \quad (3)$$

where d_j is the distance between index station j and the interpolation station, and n is the number of index stations used for the estimation. It has been concluded (Shearman and Salter, 1975; Wei and McGuinness, 1975; Dean and Snyder, 1977) that $p=2$ better approximates the isohyetal map drawn by conventional methods.

Normal Ratio Method (NR) --

A missing value y_t is estimated as:

$$y'_t = \frac{1}{n} \sum_{j=1}^n \left(\frac{R}{R_j} x_{j,t} \right) \quad (4)$$

where R and R_j are the normal annual or monthly rainfall at the interpolation and index station j respectively. This method is recommended (Paulhus and Kohler, 1952) when the normal rainfall at any of the index stations differs from that of the interpolation station by more than 10 percent.

Modified Weighted Average Method (MWA) --

The RD and NR methods may be both written in the general form of a weighted average scheme:

$$y'_t = A X_t \quad (5)$$

where A is a row vector ($1 \times n$) and X_t is a column vector ($n \times 1$). To preserve the mean, $\bar{y}$, and variance, s_y^2 , estimated from the available data, a modified scheme may be used:

$$y'_t = B X_t + (\bar{y} - \bar{B} X) \quad (6)$$

where

$$B = A \frac{s_y}{s_{y'}} \quad (7)$$

and $s_{y'}$ is obtained from:

$$s_{y'}^2 = A \text{cov}[X] A^T = \sum_{i=1}^n \sum_{j=1}^n a_i a_j c_{ij} \quad (8)$$

where c_{ij} is the covariance between elements x_i and x_j of the rainfall series of the index stations i and j (Kottegoda and Elgy, 1977; Foufoula-Georgiou, 1982).

Comparison of the Methods

Evaluation of the methods is based on the statistical comparison of the estimated series (mixture of existing and estimated values) to the incomplete series (what is really available in practice) and to the actual series (unknown in practice but known in this artificial case).

The following notation is introduced:

- $\bar{y}_e, s_e, r_e$ = mean, standard deviation and serial correlation coefficient of the estimated series;
- $\bar{y}_i, s_i, r_i$ = same as above but for the incomplete series, where $i = 1, 2, 3, 4$ and 5 for the five different percentages of missing values;
- $\bar{y}_a, s_a, r_a$ = parameters of the actual series;
- $\bar{y}_r$ = mean of the residuals (estimated - actual values);
- $s_r^2, s_{r,e}^2$ = variance of the residuals over the whole series and over only the estimated values respectively.

The criteria used for the comparison of the methods are:

- (1) the bias in the mean as measured by
(a) $\bar{y}_e - \bar{y}_i$ and (b) $\bar{y}_e - \bar{y}_a$;
- (2) the bias in the standard deviation as measured by
(a) s_e/s_i and (b) s_e/s_a ;
- (3) the bias in the serial correlation coefficient as measured by $r_e - r_a$;
- (4) the bias of the estimation model as given by the mean of the residuals, $\bar{y}_r$ (this is also a way to detect a consistent over- or under-estimation by a method);
- (5) the accuracy as determined by the variance of the residuals s_r^2 and $s_{r,e}^2$;
- (6) the significance of the biases in the mean, standard deviation and correlation coefficient as determined by the appropriate test statistic for each.

Regarding comparison of the means the following can be concluded from Table 1:

- (1) the bias in the mean in all cases is not significant at the 5% significance level as shown by the appropriate t-test;
- (2) the bias in the mean of the incomplete series is relatively small but becomes larger the higher the percent of missing values;
- (3) at high percents of missing values the NR method gives the less biased mean;
- (4) except for the RD method which consistently overestimates the mean (the bias being larger the higher the percent of missing values), the other methods do not show a consistent over or underestimation.

Regarding comparison of the variances the following can be concluded from Table 2:

- (1) although slight, the bias in the standard deviation is always significant, but this is so because the ratio of variances would have to equal 1.0 exactly to satisfy the F-test (i.e., be unbiased) with as large a number of degrees of freedom as in this study;
- (2) the MV method always gives a reduced variance as compared to the variance of the incomplete series and of the actual series, the bias being larger the higher the percent of missing values;
- (3) the bias in the standard deviation of the incomplete series is small;
- (4) there is no consistent over or under-estimation of the variance by any of the methods (except the MV method);
- (5) the MWA method does not give less biased variance even at the higher percent of missing values tested, as compared to the RD and NR methods.

Regarding comparison of the correlation coefficient the following can be concluded from Table 3:

- (1) the bias in the correlation coefficient is in all cases not significant at the 5% significance level as shown by the appropriate z-test;
- (2) the MV method gives the largest bias in the correlation coefficients, the bias increasing the higher the percent of missing values;
- (3) all methods (except the MWA method) consistently overestimate the serial correlation coefficient of the incomplete series but not the serial correlation of the actual series, and therefore this is not considered

Table 1. Bias in the Mean.

	INC	MV	RD	NR	MWA	
	$(\bar{y}_e - \bar{y}_i)$					$\bar{y}_i$
2%	0.	0.009	0.008	0.002	0.003	4.116
5%	0.	-0.012	0.014	-0.008	0.003	4.113
10%	0.	-0.010	0.006	-0.024	-0.017	4.144
15%	0.	-0.089	0.042	0.000	-0.001	4.135
20%	0.	0.042	0.149	0.043	0.086	4.082
	$(\bar{y}_e - \bar{y}_a)$					$\bar{y}_a$
2%	-0.010	-0.001	-0.002	-0.012	-0.013	4.126
5%	-0.013	-0.025	0.001	-0.021	-0.010	
10%	0.018	0.008	0.024	-0.006	0.001	
15%	0.009	-0.020	0.051	0.009	0.008	
20%	-0.044	-0.002	0.105	-0.001	0.042	

Table 2. Bias in the Standard Deviation.

	INC	MV	RD	NR	MWA	
	s_e/s_i					s_i
2%	1.	0.995	0.998	0.996	0.998	3.680
5%	1.	0.983	1.007	1.001	1.013	3.671
10%	1.	0.972	0.996	0.986	1.005	3.705
15%	1.	0.957	0.988	0.978	0.994	3.671
20%	1.	0.944	1.006	0.973	1.011	3.701
	s_e/s_a					s_a
2%	1.002	0.997	1.000	0.998	1.000	3.673
5%	0.999	0.983	1.006	1.000	1.013	
10%	1.009	0.981	1.004	0.994	1.014	
15%	0.999	0.956	0.988	0.978	0.994	
20%	1.008	0.952	1.014	0.980	1.019	

Table 3. Bias in the serial correlation coefficient.

	INC	MV	RD	NR	MWA	
	$(r_{1,e} - r_{1,a})$					$r_{1,a}$
2%	--	0.005	0.001	0.002	-0.003	0.366
5%	--	0.006	0.003	0.001	-0.002	
10%	--	0.013	0.014	0.011	0.010	
15%	--	0.033	0.006	0.013	-0.009	
20%	--	0.042	0.004	0.011	-0.012	

Table 4. Accuracy--Mean and Variance of the Residuals.

N = total number of values = 660

 N_o = number of missing values

	INC	MV	RD	NR	MWA	
	$r = (y_e - y_a)/N_o$					N_o
2%	--	-0.043	-0.061	-0.570	-0.589	13
5%	--	-0.440	0.034	-0.380	-0.176	33
10%	--	0.007	0.156	-0.113	-0.046	62
15%	--	-0.175	0.338	0.074	0.105	98
20%	--	0.037	0.502	0.038	0.200	130
	$s_{r,e}^2 = (y_e - y_a)^2/(N_o - 2)$					
2%	--	5.037	2.874	3.149	4.585	
5%	--	8.610	3.656	3.411	5.340	
10%	--	7.892	4.239	3.484	5.187	
15%	--	7.620	4.630	3.958	5.816	
20%	--	5.224	4.891	3.681	4.898	
	$s_r^2 = (y_e - y_a)^2/(N - 2)$					
2%	--	0.084	0.048	0.053	0.077	
5%	--	0.406	0.172	0.161	0.252	
10%	--	0.720	0.387	0.318	0.473	
15%	--	1.112	0.675	0.577	0.849	
20%	--	1.016	0.951	0.716	0.953	

- a problem;
- (4) the RD method seems to give the less biased correlation coefficient even at the higher percentage of missing values.

Regarding accuracy of the methods the following can be concluded from Table 4:

- (1) no method seems to consistently over or underestimate the missing values at all percent levels, but at high percent levels the missing values are overestimated by all methods;
- (2) the NR method is the most accurate method especially at high percents of missing values (i.e., it gives the smallest mean and variance of the residuals).

ESTIMATION BY A UNIVARIATE STOCHASTIC MODEL

Introduction

The observed monthly rainfall series, y_t , is normalized using the square root transformation (Roesner and Yevjevich, 1966; Stidd, 1970; Delleur and Kavvas, 1978) and the periodicity is removed by subtracting the monthly means and dividing by the standard deviations (Kavvas and Delleur, 1975). The reduced series, z_t , approximately normal and stationary is then modeled by an ARMA(1,1) model:

$$z_t = \phi z_{t-1} - \theta \alpha_{t-1} + \alpha_t \quad (9)$$

where ϕ, θ are the autoregressive and moving average parameters respectively, and α_t is a sequence of independent random variables from a normal distribution with zero mean and unit variance (white noise).

For an ARMA(1,1) model the minimum mean square error forecasts $z'_t(\ell)$ of $z_{t+\ell}$, where ℓ is the lead time are:

$$z'_t(\ell) = \phi z_t - \theta \ell_t, \quad \ell=1 \quad (10)$$

$$z'_t(\ell) = \phi z'_t(\ell-1), \quad \ell=2, \dots, k$$

as developed by Box and Jenkins (1976).

Proposed Estimation Algorithm

The estimation of the missing values in the series is performed recursively by the following procedure:

Step 1: The incomplete series $\underline{S}_0$ is filled-in with any initial estimates of the missing values giving the complete series, $\underline{S}_1$.

Step 2: An ARMA(1,1) model is fitted to the series $\underline{S}_1$ and the maximum likelihood estimates (MLE) of the parameters ϕ and θ are found.

Step 3: New estimates of the missing values are calculated as forecasts of the model $\underline{M}_1 = (\phi, \theta)_1$ (made at the appropriate origin for each gap and using equation 10). The new estimates replace the old ones and the series becomes $\underline{S}_2$.

Steps 2 and 3 are repeated as many times as needed until convergence is obtained in the sense that no significant change occurs in the estimates of the

missing values as well as in the parameters of the model.

The above algorithm will be addressed as RAEMV-U (Recursive Algorithm for the Estimation of Missing Values - Univariate model) and is schematically shown in Fig. 2. A FORTRAN program has been developed for the above algorithm (Foufoula-Georgiou, 1982). Input is the incomplete rainfall series and the positions of the gaps. Output is the final estimated complete series as well as the final parameters of the fitted ARMA model.

Results of the Method

Little influence of the method used to determine initial estimates of missing values was found on the final values of parameters ϕ and θ and on the final estimates of missing values computed by the recursive scheme. All methods that were tried yielded identical estimates of missing values and model parameters after five iterations at 10 percent missing values ($\phi = 0.5095$, $\theta = 0.4333$) and eight iterations at 20 percent missing values ($\phi = 0.0776$, $\theta = -0.0293$). Moreover, by using zeroes as initial estimates the same results were obtained, suggesting the latter as a convenient choice.

The RAEMV-U method was assessed using the same statistical measures as used for the four traditional techniques described previously. Table 5 shows the bias in the mean, standard deviation and serial correlation coefficient for the final series (at 10% and 20% missing values). The bias in the mean and correlation coefficient is not significant at the 5% significance level; however, the bias in the standard deviation does not pass the stringent F-test (requiring exact equality of standard deviations) and thus is significant.

Table 5. Bias in the Mean, Standard Deviation and Serial Correlation Coefficient-Univariate Model.

	$\bar{y}_e - \bar{y}_a$	s_e/s_a	$r_{1,e} - r_{1,a}$
10%	-0.021	0.983	0.018
20%	-0.083	0.951	0.044

The forward mean square error forecasting procedure that was used worked satisfactorily in the sense that rapid convergence to a statistically acceptable series occurred. Damsleth (1980) introduced the optimal between-forecasts as that linear combination of forecasts and backforecasts which gives the minimum mean square error. For the case of monthly rainfall data the use of more sophisticated forecasts seems not to be justified. The parameters ϕ and θ of the fitted ARMA(1,1) model are very close to each other and the value of ϕ is small as compared to one, thus making the large white noise variance the predominant term in the calculation of the mean square forecast error (Box and Jenkins, 1976, p. 154).

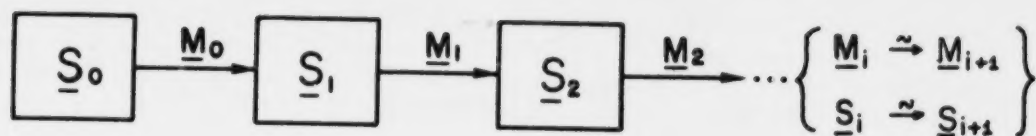


Fig. 2. Recursive Algorithm for the Estimation of Missing Values--Univariate model (RAEMV-U). $\underline{S}_i$ denotes the series, and $\underline{M}_i$ the model, $(\phi, \theta)_i$, at the i th iteration.

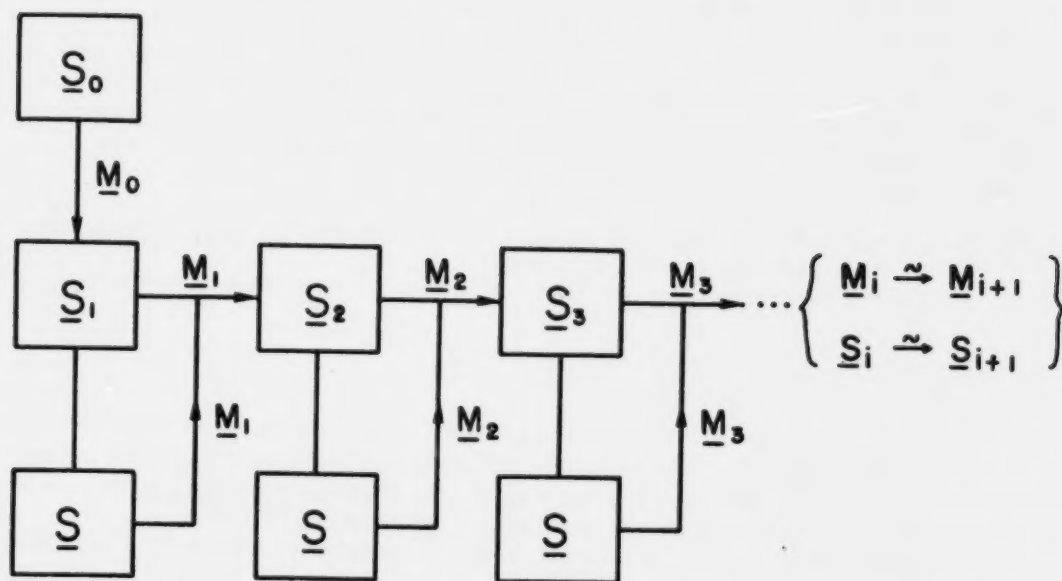


Fig. 3. Recursive Algorithm for the Estimation of Missing Values--Bivariate model (RAEMV-B). $\underline{S}_i$ denotes the series, and $\underline{M}_i$ the model, $(P, Q)_i$, at the i th iteration.

ESTIMATION BY A MULTIVARIATE STOCHASTIC MODEL

Introduction

When the concurrent rainfall series of nearby stations are available, their correlation with the series of interest may be incorporated in the model for an improved estimation of the missing values. The lag-one multivariate autoregressive model (Matalas, 1967) is expressed as:

$$Z_t = P Z_{t-1} + Q H_t \quad (12)$$

where Z_t and Z_{t-1} are n -length vectors of the normalized and standardized variables at time t and $t-1$, H is an n -length vector of random components and n is the number of stations used. The above model preserves the lag-zero (M_0) and lag-one (M_1) correlation matrices when the coefficient matrices P and Q are estimated by:

$$P = M_1 M_0^{-1} \quad (13)$$

$$Q Q^T = M_0 - M_1 M_0^{-1} M_1^T \quad (14)$$

Equation (14) may be solved for Q using a principal component analysis (Fiering, 1964) or much easier by an upper triangularization technique (Young, 1968; Young and Pisano, 1968). Missing values in any of the records may result in no solution at all or a solution that contains complex numbers since $Q Q^T$ may not be a positive semidefinite matrix as required for a real solution to occur (Valencia and Schaake, 1973; Slack, 1973).

The special case considered here is that of a bivariate AR(1) model between the interpolation station A and the index station 2 (Fig. 1). This model is written as:

$$\begin{bmatrix} z_{1,t} \\ z_{2,t} \end{bmatrix} = \begin{bmatrix} p_{11} & p_{12} \\ p_{21} & p_{22} \end{bmatrix} \begin{bmatrix} z_{1,t-1} \\ z_{2,t-1} \end{bmatrix} + \begin{bmatrix} q_{11} & 0 \\ q_{21} & q_{22} \end{bmatrix} \begin{bmatrix} \eta_{1,t} \\ \eta_{2,t} \end{bmatrix} \quad (15)$$

Following the Box-Jenkins forecasting procedure, the mean square error forecasts $z'_{1,t}(\ell)$ of $z_{1,t+\ell}$ are:

$$\begin{aligned} z'_{1,t}(\ell) &= p_{11} z_{1,t} + p_{12} z_{2,t}, & \ell &= 1 \\ z'_{1,t}(\ell) &= p_{11} z'_{1,t}(\ell-1) + p_{12} z_{2,t}(\ell-1), & \ell &= 2, 3, \dots, k \end{aligned} \quad (16)$$

where k is the number of values missing in each gap.

Proposed Estimation Algorithm

An algorithm analogous to the one for the univariate case is also proposed for the bivariate case. The procedure is exactly the same, except that now the parameters of the model, $\underline{M} = (P, Q)$, are matrices calculated from equations (13) and (14), and the forecasts are calculated from eqn. (16)

The algorithm will be addressed as RAEMV-B (B stands for Bivariate model) and is shown schematically in Fig. 3. A FORTRAN program is also available (Foufoula-Georgiou, 1982). Input data are: the incomplete series of the interpolation station, the position of its gaps, and the complete series of the index station. Output results are: the final estimated complete series of the interpolation station, the parameters P and Q of the fitted bivariate model and the correlation matrices M_0 and M_1 .

Results of the Method

Again, the scheme converges rapidly and independently of the method used to obtain initial estimates of missing values, thus suggesting their convenient replacement by zeroes to start the algorithm. Also, the convergence of the bivariate scheme seems to be less sensitive to the percentage of missing values as compared to the univariate one (three to four iterations were needed in both the 10% and 20% missing values).

Table 6 shows the bias in the mean, standard deviation and serial correlation coefficient for the final series (at 10% and 20% missing values). Again, the bias in the mean and correlation coefficient is not significant at the 5% significance level, but the bias in the standard deviation is.

Table 6. Bias in the Mean, Standard Deviation and Serial Correlation Coefficient--Bivariate Model.

	$\bar{y}_e - \bar{y}_a$	s_e/s_a	$r_{1,e} - r_{1,a}$
10%	-0.030	0.983	0.016
20%	-0.049	0.959	0.050

CONCLUSIONS

On the basis of the monthly rainfall data from the four south Florida stations used in the analysis, the following conclusions can be drawn:

- (1) All the traditional estimation techniques give unbiased (overall and monthly) means and correlation coefficients at the 5% significance level even for as high as 20% missing values.
- (2) At high percentages of missing values (greater than 10%) the MV method gives the more biased (although not significantly so) correlation coefficients.
- (3) All methods give a slightly biased overall variance but unbiased monthly variance at the 5% significance level, and the MV method gives the most biased variances for all percentages of missing values.
- (4) The NR method gives the most and the MV the least accurate estimates, at almost all levels of percent missing values.
- (5) The proposed recursive algorithm works satisfactorily in both the univariate and bivariate case. It converges rapidly and independently of the initial estimates and gives unbiased means and correlation coefficients at the 5% significance level.

- (6) The use of a bivariate model as compared to a univariate one did not improve the estimates except for a slight improvement at 20% missing values. However, the use of a multivariate model based on three or four nearby stations is expected to give much better estimates. The use of three adjacent stations is the main reason for the better performance of the NR method over the more sophisticated univariate and bivariate ARMA models which use only zero and one additional stations.

If the purpose of estimation is to calculate the historical statistics of the series (e.g., mean, standard deviation, and autocorrelations) the selection of the method matters little, and the simplest one may be chosen. However, if it is desired to fit an ARMA model to the incomplete series, to be used, say, to construct forecasts, the estimation of the missing values and the parameters of the model by the proposed recursive algorithm is recommended. In this case the equilibrium state (i.e., final series and parameters of the model) achieved upon convergence is unique, depending only on the existing information in the system (available data) and not on any external information added to the system (by the replacement of the missing values with some estimates derived by an arbitrary chosen method).

ACKNOWLEDGEMENTS

I would like to acknowledge with gratitude Professor Wayne C. Huber for the invaluable guidance and encouragement he provided along the course of this research.

The study was supported in part by the South Florida Water Management District.

REFERENCES

- Box, G.E.P., and Jenkins, G.M., 1976, Time Series Analysis Forecasting and Control, Holden-Day, San Francisco, Revised ed.
- Damsleth, E., 1980, "Interpolating missing values in a time series," Scand. J. Stat., 7:33-39.
- Dean, J.D., and Snyder, W.M., 1977, "Temporally and areally distributed rainfall," J. of the Irrigation and Drainage Div., ASCE, 103(IR2):221-229.
- Delleur, J.W., and Kavvas, M.L., 1978, "Stochastic models for monthly rainfall forecasting and synthetic generation," J. Appl. Meteor., 17(10):1528-1536.
- Fiering, M.B., 1964, "Multivariate technique for synthetic hydrology," J. Hydraul. Div., ASCE, 90(HY5):43-60.
- Fiering, M.B., 1968, "Schemes for handling inconsistent matrices," Water Resour. Res., 4(2):291-297.
- Foufoula-Georgiou, E., 1982, Estimation of Missing Values in Monthly Rainfall Series, Masters Thesis, University of Florida, Gainesville.
- Kavvas, M., and Delleur, J., 1975, "Removal of Periodicities by differencing and monthly mean subtraction," J. Hydrol., 26:335-353.
- Kottegoda, N.T., and Elgy, J., 1977, "Infilling missing flow data," in Modeling Hydrologic Processes, Ed. by Morel-Seytoux, H., Salas, J.D., Sanders, T.G., and Smith, R.E., Water Resour. Res. Publications, Fort Collins, Colorado.
- Matalas, N.C., 1967, "Mathematical assessment of synthetic hydrology," Water Resour. Res., 3(4):937-945.
- Paulhus, J.L.H., and Kohler, M.A., 1952, "Interpolation of missing precipitation records," Mon. Weather Review, 80:129-133.
- Roesner, L.A., and Yevjevich, V., 1966, "Mathematical models for time series of monthly precipitation and monthly runoff," Hydrology paper No. 15, Colorado State University, Fort Collins, Colorado.
- Slack, J.R., 1973, "I would if I could (self-denial by conditional models)," Water Resour. Res., 9(1):247-249.
- Shearman, R.J., and Salter, P.M., 1975, "An objective rainfall interpolation and mapping technique," Hydrological Sciences Bulletin, 20(3):353-363.
- Stidd, C.K., 1970, "The nth root normal distribution of precipitation," Water Resour. Res., 6(4):1095-1103.

- Valencia, D.R., and Schaake, J.C., Jr., 1973, "Disaggregation processes in stochastic hydrology," Water Resour. Res., 9(3):580-585.
- Wei, T.C., and McGuinness, J.L., 1973, "Reciprocal distance squared method, a computer technique for estimating areal precipitation," ARS NC-8, U.S. Dept. of Agriculture, Washington, D.C.
- Young, G.K., 1968, "Discussion of 'Mathematical assessment of synthetic hydrology' by N.G. Matalas," Water Resour. Res., 4(3):681-682.
- Young, G.K., and Pisano, W.C., 1968, "Operational hydrology using residuals," J. Hydr. Div., ASCE, 94(HY4):909-923.

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

AREAL INTENSITY-DURATION-FREQUENCY CURVES - A POSSIBLE WAY OF IMPROVING THE RAINFALL INPUT.

by Janusz Niemczynowicz *

1. INTRODUCTION

Intensity-duration-frequency (i-d-f) relationships, usually derived from point rainfall measurements, have for a long time been used for synthesizing so-called "design storms". Simple block rain can easily be derived from i-d-f curves for the desired duration and return period and then be used as an input for simulation of runoff occurrences.

Since i-d-f curves comprise the statistical properties of a long time series of rainfall data in a comprehensive form, it is easy to believe that rainfall input derived from them has a good statistical justification. As long as the rational method was used, no proof could be found that statistical information taken from point i-d-f curves was not always sufficient for design purposes. Later on, when more sophisticated methods of runoff prediction came into common use, it was realised that the single block rain not only gives the wrong picture of a hyetograph, but also the wrong rainfall volume. (Arnell 1982)

Furthermore, it was soon realised that the rainfall frequency given by intensity-duration relationships not correspond to the observed runoff frequency. (Sieker 1978, Urbonas 1979, James 1981)

During recent years a number of design storms with different shapes have been developed. (Keifer et al 1957, Amerocho 1981, Arnell 1982) Some of them tried to reproduce a real shape of observed hyetographs. But these design storms can not reflect the dynamics of the moving storms and by no means represent areal properties of the rainfall pattern.

The main reason that there is no simple linear relationship between the frequencies of rainfall and runoff is probably the fact that the rainfall frequency comes from point observations, while runoff represents the areal and dynamical properties of the rainfall. In other words, a similar rainfall hyetograph observed by one gauge can give a number of different runoff occurrences, or vice versa.

Convective storms, which are most significant for design purposes, are rather limited in space. Design storms derived from point i-d-f curves have no areal dimensions, but are nevertheless used for runoff simulations on catchments of different sizes. We can reduce this idea "ad absurdum" by trying to imagine one single design storm falling simultaneously on thousands of square kilometers.

* Department of Water Resources Engineering,
University of Lund, Sweden.

It is obvious that some kind of reduction factors, taking into consideration the areal properties of a rainfall, have to be used, especially while modeling runoff from large catchments.

The comprehensiveness and convenience of the statistical information in i-d-f curves, make us to believe that design storms will stay with us for some time.

One possible way of improving the rainfall input derived from i-d-f curves is to develop AREAL INTENSITY-DURATION-FREQUENCY curves. Another way is to develop area-rainfall depth relationships from which factors reducing rainfall from point to areal values can be obtained for different areas, durations and return periods. Those factors can then be used to reduce the design storms from point to areal values.

These solutions are more practical than complete, because neither areal i-d-f curves nor areal reduction factors describe the dynamics of the moving storms.

The third and most desirable solution is to develop a statistical model simulating a rainfall series taking into account temporal, spatial and dynamical variations of the rainfall pattern. (Bras et al 1976, Amorochio et al 1977, Gupta et al 1979)

The need for reasonable rainfall input for runoff simulations in big cities has caused a number of area-rainfall depth relationships to be developed in different countries. (Abraham et al 1976, Bell 1976, Rodrigues-Iturbe et al 1974) One important problem to face is the lack of sufficiently long time series of observations on a dense network with good time synchronisation.

The raingauge network installed in Lund in 1978 covers approximately a 25 square kilometer area with 12 gauges. More than three years of registered data is assumed to be sufficient to produce reasonably good statistics for short term rainfalls. This paper describes point and areal i-d-f curves derived for the city of Lund. Presented areal relationships will give more realistic design storms in comparison with design storms derived from point relationships.

2. THE GAUGING SYSTEM AND DATA PROCESSING

Twelve automatic tipping-bucket gauges were installed in Lund in 1978 to cover an area of approximately 25 sq km. The depth resolution of the gauges is 0.035 mm per tipping, the time resolution of registration is one minute. All gauges are connected via open telephone lines to the receiving station in the laboratory of the department. Since all gauges are governed by the same clock, the absolute time synchronisation is achieved. Figure 1. shows the situation of the raingauges in Lund. The gauging system and the data collecting procedure have been described before. (Falk et al 1979, Niemczynowicz et al 1981). Collected data was processed according to the flow chart shown in Figure 2.

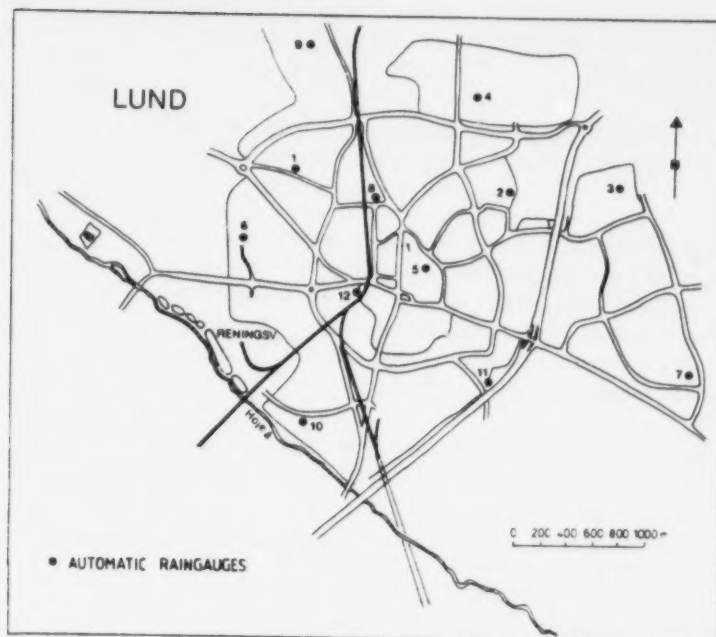


Figure 1. Situation of the raingauges in Lund.

The rainfall series from all twelve gauges were divided into rainfall events with an arbitrarily chosen interval between the events set to 40 minutes. Since the main goal of this study is to compare the point rainfall statistics with the areal rainfall statistics for short term rainfalls, all events with low intensity were taken out of the data base. The criterion chosen was that all events with rainfall depth less than 0.35 mm observed during ten successive minutes were excluded from data base. If any of the gauges exceeded the criterion, data from all other gauges were accepted for the same

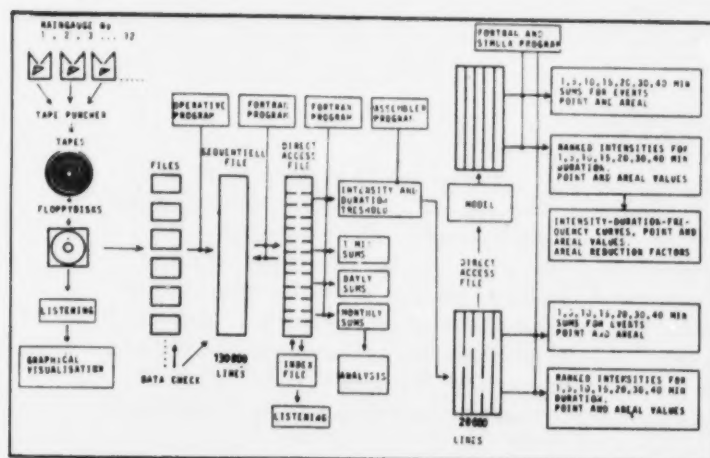


Figure 2. Flow chart for rainfall data processing.

period of time. It was found that this procedure only eliminated rainfall events with a very uniform temporal and spatial distribution. All data suspected to be wrong and all events with less than eight gauges operating were taken away from the data base. This data base, originally consisting of about 130 thousands lines was reduced to 20 thousand lines which made file operations much easier. A total of 588 rainfall events were finally included into the data base.

Malfunction of the gauges was observed or suspected about 15 % of the time on the average for the 12 gauges. Gauge No 1 was the worst with malfunctions during 38 % of the time, gauges No 2,3 and 8 were functioning all the time. Most of the malfunctions were caused by broken telephone lines.

3. POINT INTENSITY-DURATION-FREQUENCY RELATIONSHIPS

Point i-d-f curves were developed separately for all 12 gauges in Lund. The one hundred maximum values of rainfall intensity were found for durations of 1,5,10,15,20,30, and 40 minutes. Intensities for each duration were then ranked and listed for each gauge. The return period for each intensity and duration was calculated by dividing the total time of operation of the gauge by the rank number. Descending maximum intensities were then plotted on a linear scale against return periods for each duration and gauge.

Several investigators have found that the Log-Pearson typ III distribution function fits well with the maximum rainfall intensity data. (Arnell 1981) Since our main interest is the differences between point and areal rainfall, no other distribution functions were tested during this study. A Log-Pearson distribution function was fitted to the observation points by method of moments according to the computer program given by Kite. (Kite 1977)

Figure 3. shows an example of the intensity-return period diagram with observed and fitted distribution function for one of the gauges. Values from the fitted distribution curves were then rearranged and the usual form of intensity-duration-frequency curves were drawn as shown in Figure 4. The described procedure was followed for the 12 gauges and resulted in 12 complete i-d-f curves, each of them representing a point value. The mean point-value i-d-f curve was finally calculated by averaging the values for all durations and return periods. Figure 5. shows the mean point-value intensityduration-frequency curves.

4. SPATIAL VARIATIONS OF RAINFALL INTENSITY

TABLE 1. gives the range of differences in maximum intensities observed between 12 gauges in Lund. The highest (Max), the lowest (Min), the mean values (Mean), the standard deviations between 12 gauges (s) and the coefficients of variation ($CV = s/\text{mean}$) are given for different durations and return periods.

The differences between the highest and the lowest values and standard deviations decrease quickly for shorter return periods. Coefficient of

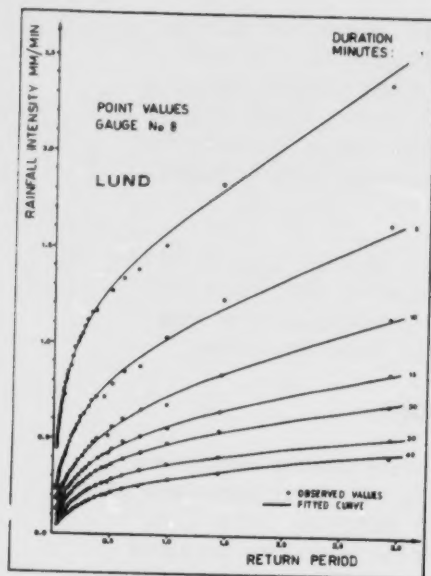


FIGURE 3. Intensity-return period diagram for gauge No 8.

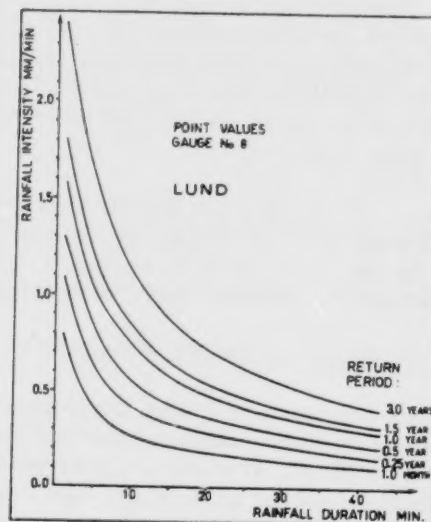


FIGURE 4. Intensity-Duration-Frequency curves for gauge No 8.

variation seems to be rather constant for all durations within the same return period, but decreases with shortening return period. Obviously, extreme intensity values are the most unevenly distributed in space. The highest values of intensity occur persistently in gauges No. 2,3,4,5 and 12 which are situated in the central part of the town and to the north-east which is the most prevailing wind direction. The lowest intensity values are typical for gauges No. 9,1,11, and 6, situated outside of the town. This effect can perhaps be explained by the influence of the city on precipitation.

5. AREAL INTENSITY-DURATION-FREQUENCY RELATIONSHIPS

In order to extrapolate rainfall data from point measurements to areal values, specific areas were associated to all gauges. The influence that different methods of extrapolating point values to areal means have on the accuracy of areal estimation was investigated by Gottshalk and Jutman (1982). Results show that the magnitude of error is to a small extent influenced by the method of extrapolation. For simplicity of calculations, the method of Thiessen polygons was chosen for this study.

Due to periodical malfunction, the gauges represented slightly different periods of observations. In order to develop areal relationships, mean rainfall values from a number of gauges had to be treated simultaneously. In order to avoid shortening the total length of the record by averaging the observation period for all gauges, a special routine for reproducing the missing values was developed.

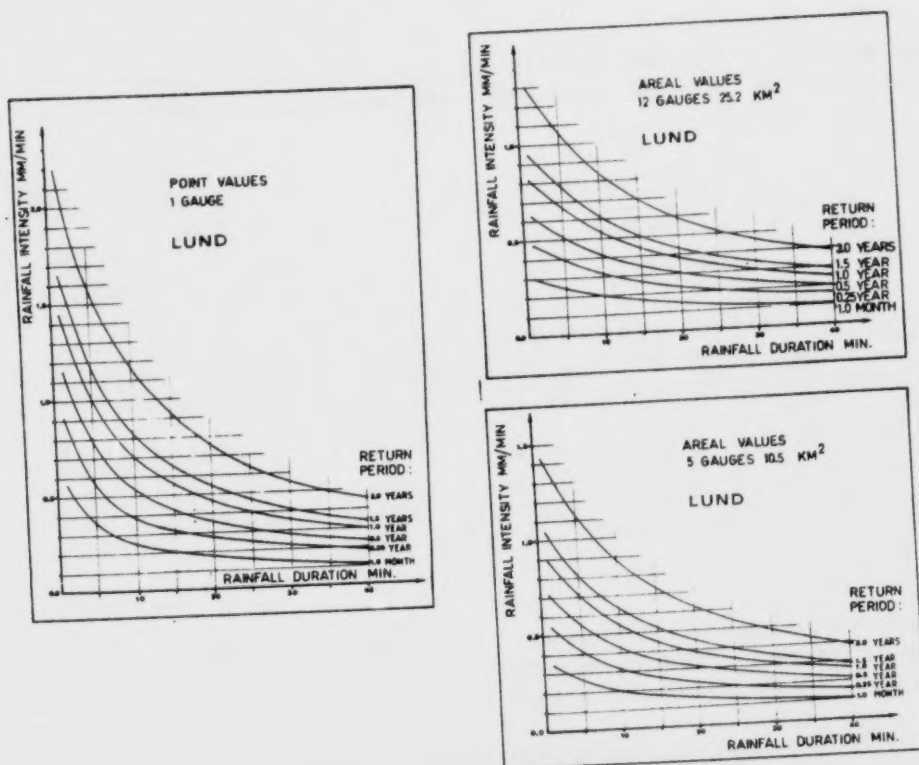


FIGURE 5. Point and areal intensity-duration-frequency curves for the city of Lund.

The average weighted values from the three nearest functioning gauges were inserted in place of the missing values for each minute of the data base. After this procedure, a new point i-d-f relationships were developed for all 12 gauges. No significant changes in results were observed after this procedure. All further calculations were performed on the same data base with reproduced missing values.

Areal i-d-f curves were developed by making calculations (described in chapter 3) on mean weighted values from combinations of the gauges. Each combination consists of 12 groups of gauges. In each combination, the same gauge is represented the same number of times. For example, a mean-value i-d-f curve representing an area of two gauges was developed as follows: mean value of rainfall intensity for each minute in the data base was calculated for the arbitrarily chosen group of two gauges.

Intensity-duration-frequency relationships were then developed for this group. The next group of two gauges was chosen and new relationships were developed. Calculations proceeded until 12 groups of gauges were treated. This resulted in 12 different, complete intensity-duration frequency relationships representing different areas of pairs of gauges. Finally, the mean-value relationship was calculated representing the mean area of 12 pairs of gauges.

TABLE 1. MAXIMUM RAINFALL INTENSITIES OBSERVED IN 12 GAUGES IN LUND DURING A THREE YEAR PERIOD. (The number of the gauge where the value was observed is in parenthesis.)

MAXIMUM		D U R A T I O N						I N		M I N U T E S	
INTENSITY											
MM/MIN	1	5	10	15	20	30	40				

RETURN PERIOD 3.0 YEARS											
Max	3.19(7)	1.98(2)	1.36(3)	1.00(2)	0.84(3)	0.62(3)	0.55(4)				
Min	1.31(9)	1.08(11)	0.86(9)	0.64(9)	0.51(9)	0.40(9)	0.37(9)				
Mean 12											
gauges	2.20	1.61	1.20	0.93	0.76	0.57	0.47				
s	0.61	0.51	0.43	0.34	0.26	0.18	0.15				
CV	0.27	0.24	0.26	0.32	0.37	0.41	0.37				

The same procedure for calculating was carried on for the groups consisting of 3,4,5,6,8,10 and 12 gauges. Intensity-duration-frequency curves for point value, and the 10, and 20 sq.km values are shown in Figure 5.

For all intensity-duration-frequency curves was fitted a mathematical expression of the form:

$$I = \frac{a}{T + b} + c$$

where I - maximum areal rainfall intensity for duration T (mm/h)

T - rainfall duration (min)

a,b,c - constants

The constants a,b, and c were calculated using the least squares criterion and the optimization routine accordingly to Marquardt's method. (Ericsson 1979).

6. AREAL VARIATIONS OF RAINFALL INTENSITY

By rearranging the values from point and areal i-d-f relationships the statistical areal reduction factors were developed for different durations and return periods. Factors for 3.0 and 1.0 years return periods are shown in Figure 6. It is interesting to notice that statistically derived areal reduction factors not only depend on duration, but also on the return period, which was questioned before. (Bell 1976)

7. CONCLUSIONS

Areal intensity-duration-frequency relationships give different rainfall values than point i-d-f relationships. Areal rainfall intensity values are lower for all durations and return periods.

If point i-d-f curves are used for deriving design storms, the error in average rainfall intensity and rainfall volume will be introduced for simulation of runoff from real catchments.

The magnitude of error depends on duration, return period and the size of the catchment.

The most significant differences between point and areal rainfall values can be found for short durations and long return periods.

Factors reducing point rainfall values to areal values are given in this paper. The presented relationships will give a more realistic design storms in comparison with such storms derived from point i-d-f curves.

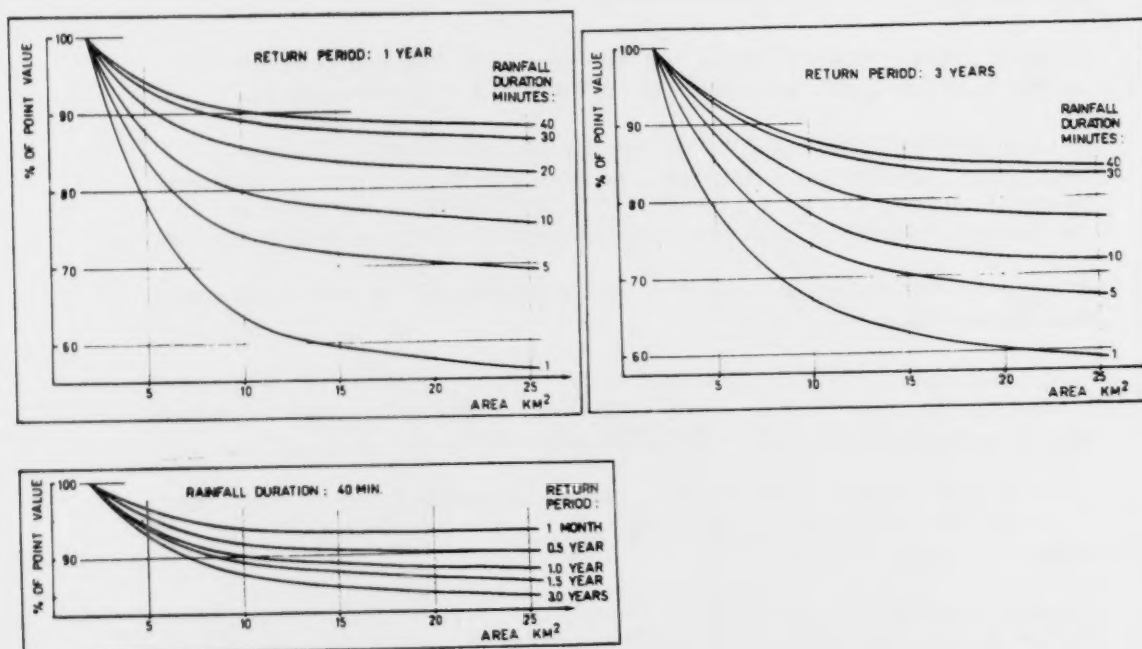


FIGURE 6. Statistical areal reduction factors.

REFERENCES

- Abracham, C., Lyons, T., C., Schulze, K., W., (1976): "Selection of a Design Storm for Use With Simulation Models". National Symposium on Urban Hydrology, Hydraulics and Sediment Control, Univ. of Kentucky, July 1976.
- Amoroch, J., Wu, B., (1977): "Mathematical Models for the Simulation of Cyclonic Storm Sequences and Precipitation Fields". Journal of Hydrology, 32 1977.
- Amoroch, J., (1981): "Stochastic Modeling of Precipitation in Space and Time". International Symposium on Rainfall-Runoff Modeling Mississippi State Univ., May 1981.
- Arnell, W., (1982): "Rainfall Data for the Design of Sewer Pipe Systems", Chalmers Institute of Technology, Report series A:8, Göteborg 1982.
- Bell, F., C., (1976): "The Areal Reduction Factors in Rainfall Frequency Estimation". Institute of Hydrology, Wallingford, U.K. Report no 39 December 1976.
- Bras, R., L., Rodriguez-Iturbe, I., (1976): "Rainfall Network Design for Runoff Prediction". Water Resources Research Vol. 12 no 6, December 1976.
- Ericsson, G., (1979): "Numerisk analys". Institutionen för informationsbehandling. Sigma-tryck, Lund 1979.
- Falk, J., Jönsson, O., Niemczynowicz, J., (1979): "Measurements of Rainfall Intensities in Lund". Lund Institute of Technology, Department of Water Resources Engineering, Report No 3023, 1979.
- Gottshalk, L., Jutman, T., (1982): "Calculation of Areal Means of Meteorologic Variables for Watersheds" Nordiske Hydrologiske Konferanse, Förde, Juni 1982.
- Gupta, V., K., Waymire, E., C., (1979): "A Stochastic Kinematic Study of Subsynchronous Space-Time Rainfall". Water Resources Research Vol. 15 No 3 June 1979.
- James, W., (1981): "Kinematic Design Storm Incorporating Spatial and Time Averaging". Second International Conference on Urban Storm Drainage, Univ. of Illinois, Urbana, June 1981.
- Keifer, C., U., Chu, H., H., (1957): "Synthetic Storm Pattern for Drainage Design". Journal of the Hydraulics Div. ASCE, Vol. 83 No Hy4, August 1957.
- Kite, G., W., (1977): "Frequency and Risk Analysis in Hydrology". Water Resources Publications, Fort Collins, Colorado 1977.
- Niemczynowicz, J., Jönsson, O., (1981): "Extreme Rainfall Events in Lund 1979-1980" Nordic Hydrology, 12, 1981.

Rodriguez-Iturbe, I., Mejia, J., M., (1974): "On the Transformation of Point Rainfall to Areal Rainfall". Water Resorces Research Vol.10 No 4 August 1974.

Sieker, F., (1977): "Statistical Simulation Model Based on Analysis of Variance". 3-rd International Hydrological Symposium, Fort Collins, June 1977.

Sieker, F., (1978): "Investigation of the Accuracy of the Postulate "Total Rainfall Frequency Equal Flood Peak Frequency"", International Conference on Urban Storm Drainage Univ. of Southampton, April 1978.

Urbonas, B., (1979): "Reliability of Design Storms in Modeling". International Symposium on Urban Storm Runoff, Univ. of Kentucky, July 1979.

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

HYDROLOGICAL REGIONALISATION : A QUESTION OF PROBLEM AND SCALE

I. Simmers and E. Seyhan : Free University, Amsterdam

Extended Abstract

Quantifying catchment hydrological response and subsequent regionalisation of resultant information have until now been largely dependent on techniques of:

- (a) generalised water balance determinations
- (b) regional statistical generation of descriptive variables or model parameters
- (c) lumped catchment models
- (d) complex process models which rely on detailed plot or experimental basin studies for input parameters.

The spectrum of available models does not reflect individual dissatisfaction with the approach of others, but is a manifestation of two fundamental issues, viz. the type of problem to be solved and the scale at which a solution is required. It is therefore to be expected that some models are ideal for initial estimates of a water resource but are unusable in an operational mode. Others present problems of translation to basins of varying size, while many have computing requirements which are so great that their practical application is restricted to research areas where economic criteria are less dominant.

The classification of an area into distinctive hydrological response units from natural resource maps is seen as a first step to resolve the obvious confusion surrounding data regionalisation. Unit identification may be achieved using multivariate statistics and, dependent on the immediate problem, can be at any scale.

Preliminary results from a continuing study in east Luxembourg are presented to demonstrate the technique. Since the problem to be resolved involves prediction of hydrological response consequent upon landuse change, the scale is small and requires application of a physically based, dynamic contributing area approach. Data collection was initiated from a regionalised sampling matrix based on topography, vegetation cover, soil and rock type.

The results from several study catchments are encouraging and adequately define the spatial variability of hydrological response and the varying influences of physiographic parameters on mechanisms which govern streamflow generation. Schematic models are presented for both hillslope processes and total system operation within a larger area. Regionalisation and solution of the present problem are thus deemed achievable by way of reconstituted response (matrix) unit flow diagrams, requiring only verification sampling.

In areas where natural resource maps are available but quantitative data are scarce, the described approach to regionalisation shows promise and will certainly minimise instrumental and data collection requirements.

Authors' address

Prof.dr. I. Simmers; Dr.ir. E. Seyhan,
Dept. Hydrogeology and Geographical Hydrology,
Institute of Earth Sciences, Free University,
P.O. Box 7161, 1007 MC Amsterdam, The Netherlands

HYDROLOGICAL REGIONALISATION : A QUESTION OF PROBLEM AND SCALE

I. Simmers and E. Seyhan : Free University, Amsterdam

ABSTRACT

Parameter prediction for ungauged catchments and regionalisation of hydrological data are bound by two fundamental issues - the 'problem' to be resolved and the 'scale' at which a solution is required. The variety of current approaches to the question suggests that there is a need to develop a simple, robust method for regionalising hydrological information which can encompass these issues. Classification of an area into distinctive hydrological response units from natural resource maps is seen as the first step to a solution; unit identification can be at any 'scale' dependent on the 'problem'. Preliminary results from a continuing study in east Luxembourg are encouraging and are presented to illustrate the technique. The 'problem' required application of a physically based, dynamic contributing area approach and data collection was initiated from a regionalised sampling matrix technique. The described approach to regionalisation is shown by this and other studies to be robust and if combined with remote sensing should minimise instrumental and data collection requirements.

INTRODUCTION

Quantifying catchment hydrological response and subsequent regionalisation of resultant information have until now been largely dependent on techniques of:

- (a) generalised water balance determinations
- (b) regional statistical generation of descriptive variables or model parameters
- (c) lumped catchment models
- (d) complex process models which rely on detailed plot or experimental basin studies for input parameters.

The spectrum of available response models does not reflect individual dissatisfaction with the approach of others, but is a manifestation of two fundamental issues, viz. the type of 'problem' to be solved and the 'scale' at which a solution is required.

For example, if a 'problem' requires generation of streamflow records from a longer rainfall time series, a lumped model may prove adequate. However, if it is necessary to predict the hydrological effects of localised landuse change, the consequences of spatially variable inputs and outputs, or the movement of pollutants and sediment through a catchment, then it is necessary to use a distributed, physically based model. Changes in catchment characteristics are thus directly reflected by changes in the model parameters (Beven and O'Connell, 1982) - such considerations cannot be satisfactorily contemplated using spatially averaging models.

Other examples and valid applications of hydrological response models defined by (a) - (d) above are of course abundantly reported in the literature and need not be elaborated on here. However, given the heterogeneity of 'problem' and 'scale' it is therefore to be expected that some models are ideal for

initial estimates of a water resource but are unusable in an operational mode. Others present problems of translation to basins of varying size, while many have computing requirements which are so great that their practical application is restricted to research areas where economic criteria are less dominant.

Furthermore, the choice of model for even one particular 'problem' is never simple - there is a marked 'contrast between the complexity of hydrological reality and the important and often pressing practicality of making management decisions based on limited knowledge of that reality' (Beven and O'Connell, *op cit*). Model selection will be based on, *inter alia*, economic constraints, hydrological considerations and data availability, with a further essential prerequisite being to match management problem requirements with the complexity of model used.

Parameter prediction from ungauged catchments and regionalisation of hydrological data are also subject to the same constraints imposed by 'problem' and 'scale'. A number of methods in current use rely on statistical regression of calibrated model parameters against catchment characteristics (*eg.* NERC, 1975), with obvious dangers. However, such an approach can resolve only a limited range of 'problems' and omits the difficulties associated with data translation at different scales due to variable storage effects.

It is evident, therefore, that there is a need to develop a simple yet robust method for regionalising hydrological information which can acknowledge these aspects. The classification of an area into distinctive hydrological response units from natural resource maps is seen as a first step to resolve the obvious confusion. Unit identification can be achieved by map interpretation and multivariate statistics and, dependent on the immediate 'problem', can be at any scale' (see, for example, Body, 1982; Krasovskaia, 1982 (a), (b); Refsgaard and Hansen, 1982; Simmers, *et al*, 1982; Seyhan and Hope, 1983).

Preliminary results from a continuing study in east Luxembourg are presented to demonstrate the technique. Since the 'problem' to be resolved involves prediction of hydrological response consequent upon landuse change, the 'scale' is small and requires the dynamic spatial variation in discharge source areas to be taken explicitly into account (Beven, *et al*, 1980; Beven and Hornberger, 1982; Freeze, 1980; Pilgrim and Bloomfield, 1980; O'Loughlin, 1981). As such, the physically based, dynamic contributing area approach adopted by Beven and Kirkby (1979) is taken as a basis for the present study. A full description of the research area, overall project objectives and analyses of data available to date are presented elsewhere (Both and van der Sommen, *in press*).

STUDY AREA

Figure 1 displays the area under investigation. Four catchments are involved thus far; two (labelled C and D) lying within Triassic Keuper formations and the others (A and B) in exposed or covered Lower Jurassic sandstone. Since the Luxembourg sandstone is more important from a local water resource point of view (*v.* Hoyer, 1971), the research project has until now focussed principally on the two catchments in this region - *viz.* Tollbach and Dosbach. The present paper reflects this priority.

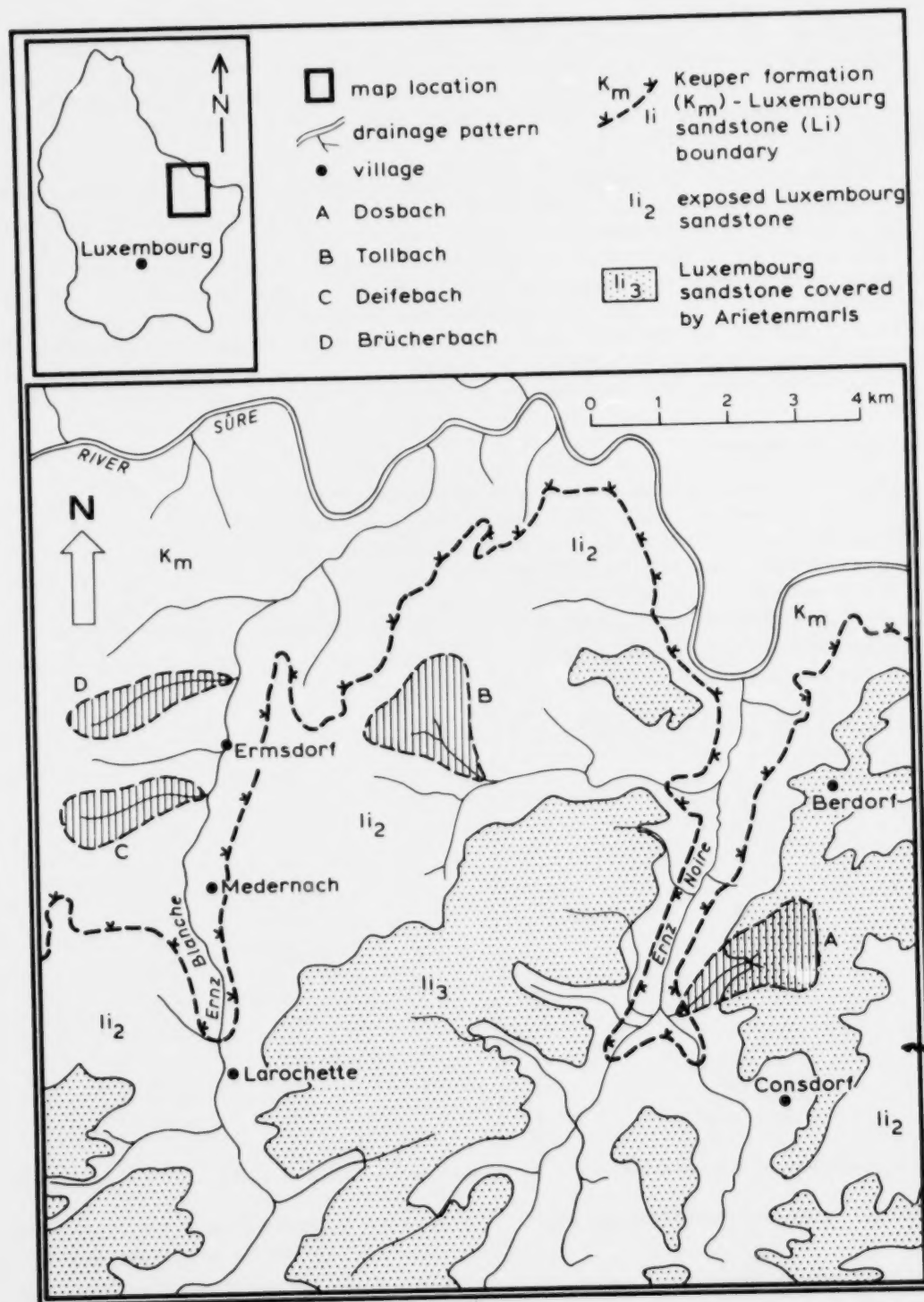


Figure 1 : East Luxembourg research area

The Tollbach lies entirely in exposed sandstone, has an area of 2.54 km² and an elevation which varies from 285 to 421 m above sea level. Conversely, weathered and unweathered Arieten marls cover 66 % of the Dosbach, and the base of the underlying sandstone is exposed in the channel close to the lowest flow measurement point. Total catchment area is 1.98 km² and elevation varies from 210 to 365 m.

The Arieten marl soils are poorly developed, though arable land areas have a well ploughed upper zone of loamy clay (~ 30 cm). Beneath this are 1-3 m of heavy, compact, blue to blue-grey clays, which cap a confined aquifer in beds of unweathered marls and karstified limestone. Weathering reduces the Luxembourg sandstone to a brown-yellow loamy sand, the clay content of which reduces both down slope and with increasing profile depth. The Luxembourg sandstone aquifer is the most important in Luxembourg and is characterised by short residence times (~ 1 year - v. Hoyer, 1971) and spring discharges which reflect, inter alia, the influences of tectonic control and degree of marl cover.

DATA NETWORK DESIGN

In order to determine the nature and dynamics of runoff production within each major geological unit data collection was initiated on a regionalised sampling matrix according to topography, vegetative cover and nature of surficial deposits, similar to that proposed by Tricker (1981). The watershed is thus regarded as a system and is considered to consist of a series of 'homogeneous' units, each playing their particular role in runoff formation.

slope position :	1	2	3	-
vegetation :	b	p	g	c
geology (soil type) :	K	S	A	-
slope position :				
1 - downslope and riparian zone				
2 - midslope				
3 - upslope				
vegetation :				
b - beech forest				
p - pine/spruce forest				
g - grassland				
c - arable land (crop)				
geology :				
K - Keuper marls				
S - Luxembourg sandstone				
A - Arieten marls				

Matrix unit No. (fig.2)	Matrix unit (table 1)	% Dosbach area	% Tollbach area
1	Kb1	0.6	-
2	Sb1	2.0	1.0
3	Sg2	0.4	-
4	Ag1	1.0	-
5	Kb2	4.5	-
6	Sb2	5.5	5.2
7	Sp2	2.0	30.9
8	Ab2	1.0	-
9	Sg2	15.5	-
10	Sc2	1.6	1.5
11	Ag2	29.7	-
12	Ac2	2.5	-
13	Ap2	4.4	-
14	Ag3	8.7	-
15	Ab3	13.7	-
16	Ap3	4.5	-
17	Sg3	1.8	-
18	Sp3	0.6	29.1
19	Sp1	-	1.0
20	Sb3	-	18.5
21	Sc3	-	12.8

Table 1 : Basic sampling matrix

Table 2 : Dosbach and Tollbach homogeneous sampling units

Table 1 gives the basic matrix used, while table 2 and figure 2 show the resultant division of the Dosbach and Tollbach into relatively homogeneous sampling units. Such a procedure obviates the necessity for excessive data

collection and the derived results may be objectively incorporated into the Beven and Kirkby (1979) model without the possibility of introduced bias. It is of interest to note that for the Dosbach eight units allow representative sampling over 87 % of the total area, and for the Tollbach this decreases to five units for 97 %.

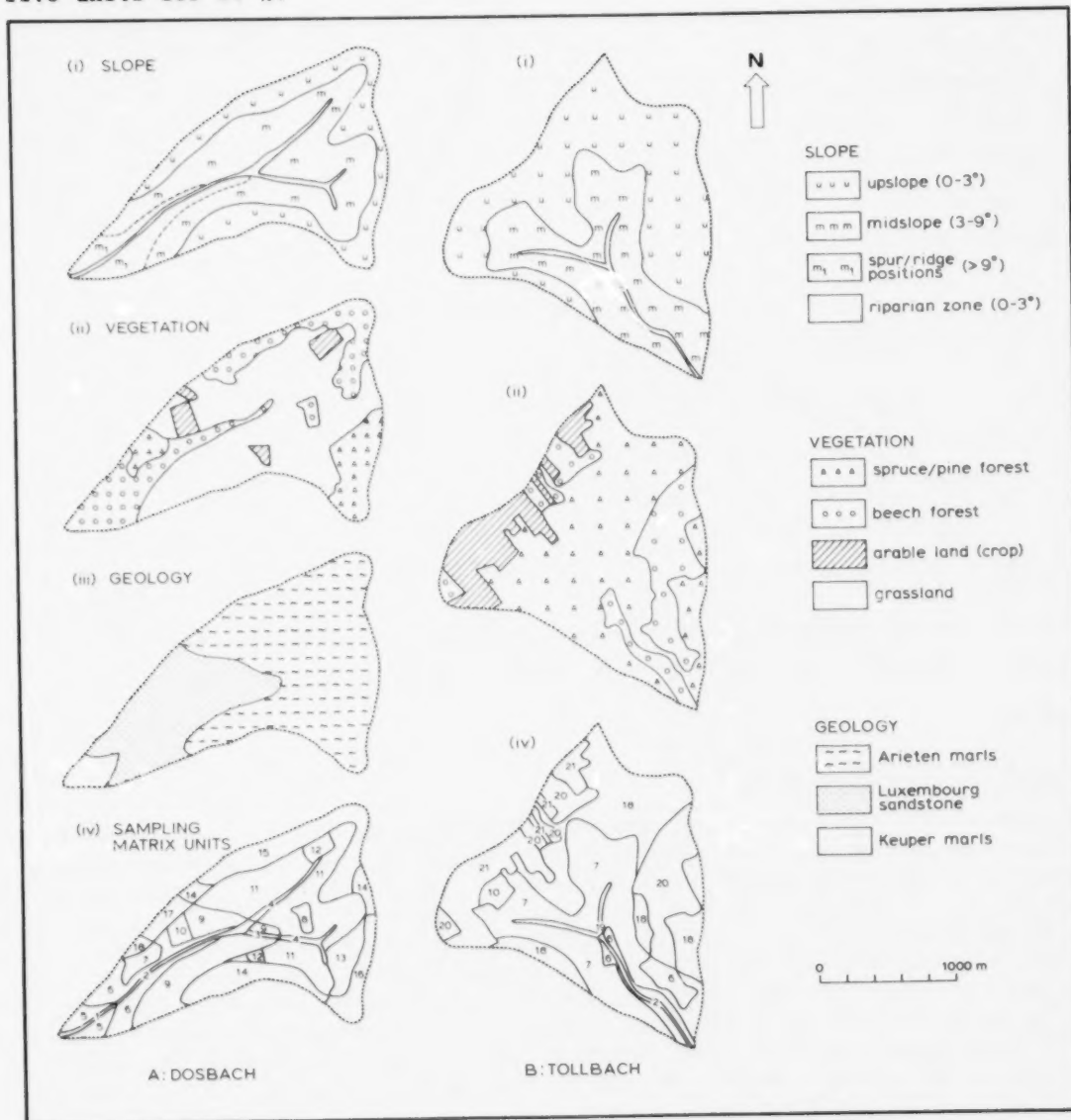


Figure 2 : Slope (i), vegetation (ii), geology (iii) and sampling matrix units (iv) for the Dosbach (A) and Tollbach (B) catchments

Data collection was largely in accordance with the indicated division and involved measurement of normal input-output variables, soil moisture levels, surface-, spring-, well- and soil-water chemistry, piezometric levels and selected soil hydraulic properties such as infiltration rate and conductivity. Primary sampling sites for the Dosbach are shown in figure 3.

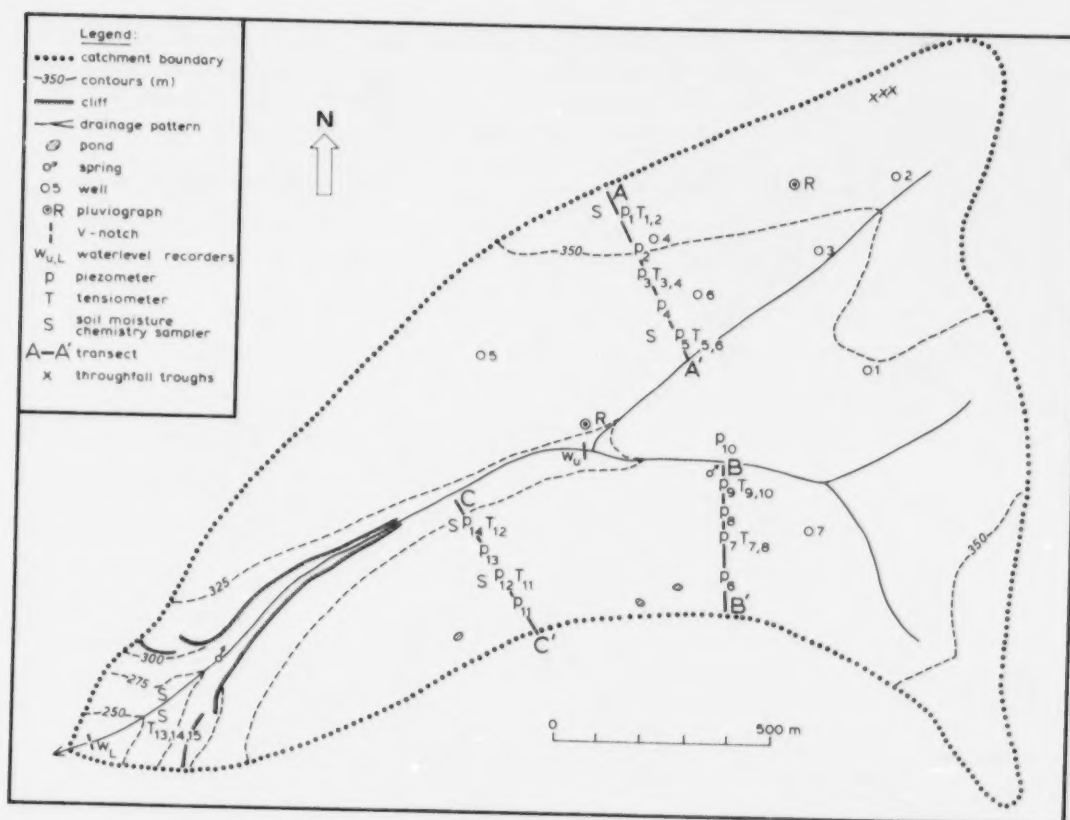


Figure 3 : Dösbach field measurement and instrumentation network

RESULTS

Streamflow characteristics

Analyses of the basic discharge and precipitation records demonstrate marked differences in response between the Tollbach and Dösbach. Groundwater from a number of distinct aquifers plays a dominant role in the total runoff process for the Tollbach, with $< 2\%$ of measured precipitation resulting in quickflow from storms with maximum rainfall intensities which vary from 2.8-20.3 mm/hr. For the Dösbach, with 66 % marl cover, mean baseflow is $\approx 45\%$ of that recorded from the Tollbach, while corresponding quickflow response varies between 5 and 60 % of storm precipitation. The overall results, when taken in conjunction with hydro-chemical investigations, confirm the appropriateness of present project objectives and methods.

Modelling the runoff process

The existence of various mechanisms for streamflow generation is now widely accepted and summaries of current knowledge are presented by, inter alia, Dunne (1978) and Freeze (1980). All recognise the importance of soil parameter heterogeneity in space, time and depth governing these mechanisms.

Unfortunately insufficient data have as yet been collected or processed to allow rigorous testing of the complete Beven and Kirkby model, so emphasis is placed here only on measurements relating to variations in soil moisture, infiltration rate, saturated hydraulic conductivity and water table depth for a number of the proposed sampling units (figure 2). Since surface processes are relatively less important in the Tollbach principal effort to date has been on data collection in the Dosbach, though sufficient information is now available to propose schematic models for both hillslope processes and total system operation in the Luxembourg sandstone region (figures 4 and 5).

Evidence to support these models is derived in part from measurement transects AA', BB' and CC' shown in figure 3. Piezometers have depths of 0.5 to 3.0 meters and tensiometer pairs are installed at 17 and 35 cm. Saturated hydraulic conductivities for varying profile depths were determined using the constant head method and infiltration parameters by sprinkling infiltrometer. Tables 3 and 4 present a sample of results obtained.

Date	Rain (mm)	A-A'								B-B'						C-C'				
		P1	T2	P2	P3	T4	P4	P5	T6	P6	P7	T8	P8	P9	T10	P11	P12	P13	P14	T12
28/9	6	8	s	11	11	s	7	73	52.0	14	7	s	6	44	50.0	42	12	-	100 cm	-
29	-	16	s	9	8	s	8	76	51.5	11	5	53.2	2	33	49.8	37	8	-		39.0
30	-	29	s	12	9	s	8	81	51.3	16	9	52.7	9	48	49.4	41	13	20		38.5
1/10	-	34	s	10	9	s	10	83	50.6	24	10	52.7	6	54	49.5	41	14	12		38.7
2	2	35	s	15	10	s	9	85	50.6	23	13	52.0	6	60	49.4	44	17	21		38.5
3	8	26	s	11	8	s	6	79	-	11	2	s	1	24	49.6	40	8	20		38.5
4	-	19	s	9	6	s	5	75	54.0	12	5	s	4	36	50.0	38	7	2		38.7
5	1	28	s	10	5	s	6	84	52.7	11	7	s	4	49	49.5	39	8	8		38.7
6	-	31	s	11	6	s	7	82	51.3	12	7	53.8	5	43	50.0	40	7	8		38.6
7	-	38	s	17	12	s	10	86	51.0	24	14	53.2	11	58	49.4	43	12	19		38.2
8	-	38	s	20	16	s	16	88	50.0	23	21	52.0	14	66	49.4	45	16	27		38.7
Matrix unit		15		11	11		11	4		14	11		11	4		14	9	9		9

P = piezometer (water level depth in cms); T = tensiometer (values at 17 cm depth expressed as soil moisture content % by volume : s (saturation) = 57%)

Table 3 : Dosbach piezometric head and soil moisture variations (28.9-8.10.81)

Matrix unit No. (fig. 2)	% Area (table 1)	Slope (°)	OFV (mm/hr)	SD (mm)	i_o (mm/hr)	vertical K_s (m/d)	lateral K_s (m/d)	K_s depth (cm)
4	1.0	-	-	-	-	20	5	20
6	5.5	25	-	7	83	-	-	-
9	15.5	5	-	33	92	3	9	20
		10	-	-	> 105			
		7	49	12	58			
11	29.7	9	-	14	51	0.1	0.007	30
		9	320*	6*	53*			
		3	170	4	43			
12	2.5	3	110	8	52	-	-	-
14	8.7	3	27	7	85	-	-	-
15	13.7	-	-	-	-	0.4	0.1	15
21	12.8	4	-	-	59	-	-	-

*= top 20 cm removed; OFV = overland flow velocity parameter; i_o = final constant infiltration rate; SD = maximum value of interception and depression store (see Beven and Kirkby, 1979)

Table 4 : Selected saturated hydraulic conductivity (K_s) and infiltration parameter estimates - (for table see previous page).

Although statistical verification of suggested interrelationships is not yet possible a number of points emerge. Allowing for minor surface irregularities water table depths reflect the dominant influences of topographic position and lithological differences, with up- and down-slope piezometers showing a strong, rapid head response during and after rain (figure 6). This is readily explained by the K_s values in table 4 despite saturated soil conditions in the upslope areas. Response is least in the midslope zone, with shallow water tables and lowest K_s .

Infiltration parameters also display considerable spatial variability, with i_o values which exceed rainfall intensities measured during the observation period. Marked differences between i_o and K_s , particularly in unit 11, suggest that despite existing vegetation cover it is unwise to assume equivalence of these parameters in areas subject to ploughing (cf. Anderson and Burt, 1978). On the Arieten marls i_o appears strongly related (but not equivalent) to only subsurface K_s , since similar i_o values are measured for varying slope angle and landuse (units 11 and 12). However, landuse has a marked influence on OFV. Conversely, K_s is found to be not a limiting factor for infiltration on weathered Luxembourg sandstone, slope angle is also unimportant, and i_o seems to be principally controlled by landuse (units 6, 9 and 21) - highest under grass cover and least with bare soil or crop as expected. The same conclusion is also reached by Tricker (1981).

DISCUSSION

The above preliminary results, supporting hydrochemical data and field observations allow a number of generalisations to be made on system operation within the Luxembourg sandstone.

At the hillslope scale on Arieten marls (figure 4) the most important runoff producing process is saturated overland flow, initiated in the midslope area (cf. Krasovskaia, 1982(a), (b)). Channel precipitation will of course contribute. Since the upper weathered zone is relatively permeable infiltration will occur until storage is filled, this occurring first in the midslope zone where water table depths and lateral K_s values are least (Freeze, 1980; O'Loughlin, 1981). Growth of the saturated area will be predominantly in a downslope direction as a result of sheet/rill flow and percolation, with subsurface flow a minimal component in this rapid response system. The small peak discharges measured in weathered sandstone areas are attributed to saturated overland flow from a narrow zone along the channel, 'Horton' overland flow on access tracks, channel precipitation and groundwater response.

Figure 5 defines the hydrological system on a regional scale. Response to precipitation is primarily controlled by the degree of marl cover, since $Q_{ma} + Q_{mf} \gg Q_{sf} + Q_{sa}$ and $Q_{bs} > Q_{bs(m)}$. Groundwater is the dominant runoff component from Luxembourg sandstone, and apart from marl cover reflects the characteristics of thickness and storage capacity of the weathered zone, structure and local lithological differences. Q_{bws} is small, as is Q_{bm} .

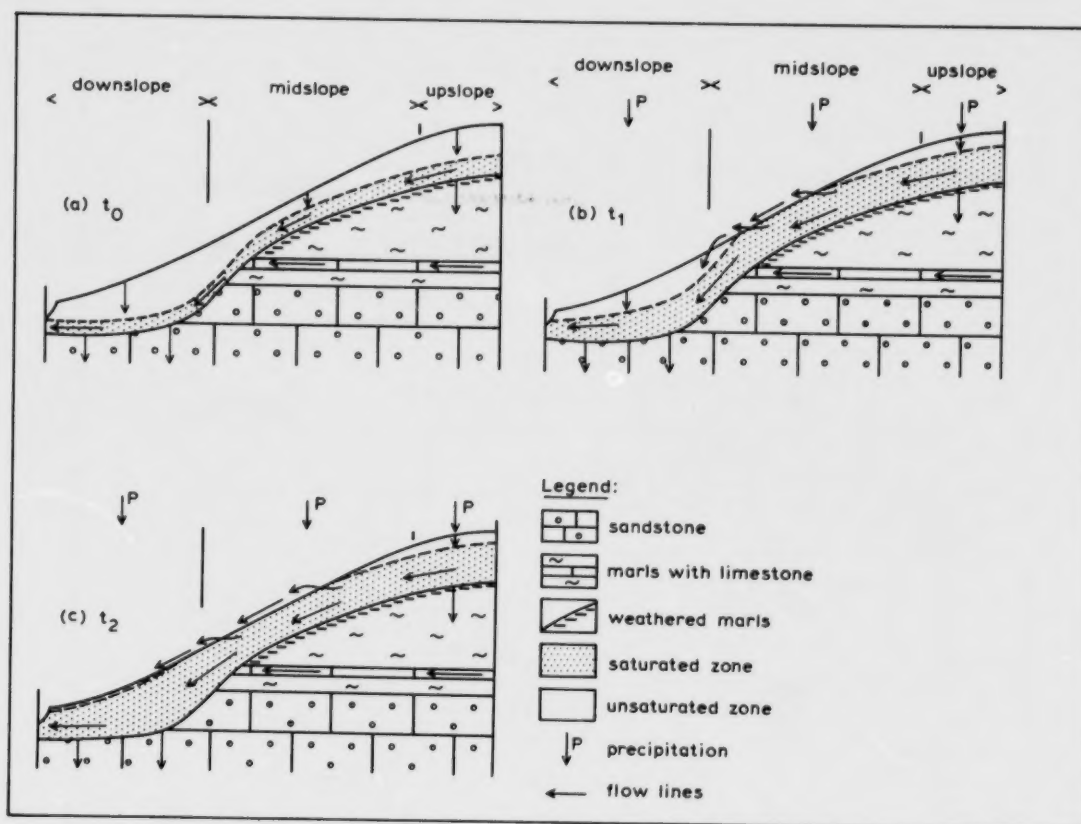


Figure 4 : Schematic model of runoff producing processes on hillslopes of the Dosbach catchment

CONCLUSIONS

Although results reported here represent only the initial investigatory phase of our ongoing programme they are a source of encouragement. The spatial variability of hydrological response is adequately defined, as are the varying influences of physiographic parameters on mechanisms which govern stream-flow generation.

To solve the immediate 'problem', testing of the Beven and Kirkby (1979) model structure under varying input and operation conditions continues and amendments will be made as necessary. Verification of model components and results is being based on further field measurements of inter- and intra-matrix unit input variability in space and time, hydrochemical and isotope studies, and geophysical surveys.

To address the more general problem posed by the paper title and introduction, the stratified sampling matrix adopted as a basis for the present overall project is found to be an appropriate technique for regionalised data collection. Regionalisation for purposes of assessing the hydrological effects of landuse change, both within the same area and elsewhere, is thus deemed achievable by way of reconstituted response (matrix) unit flow dia-

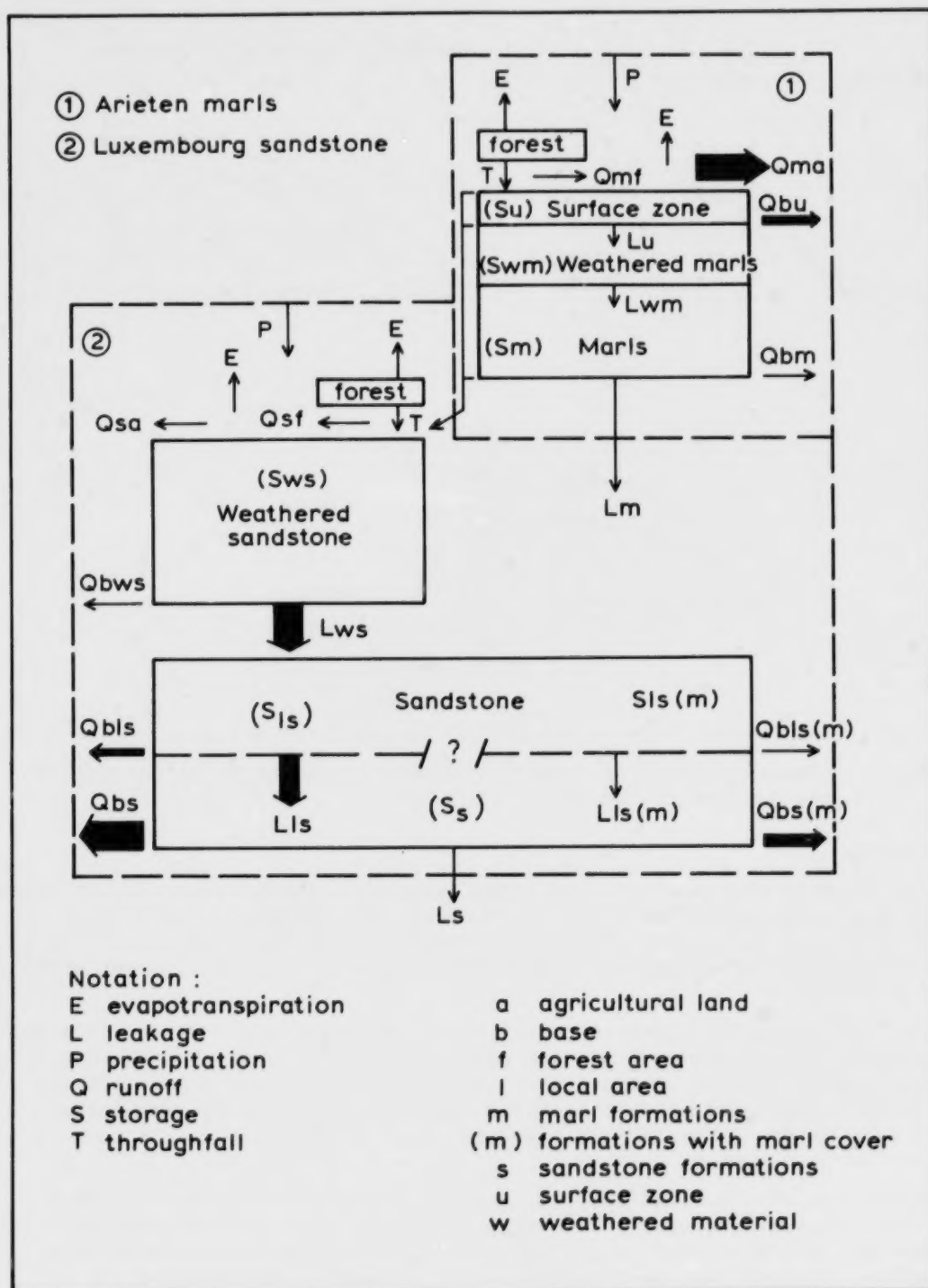


Figure 5 : Hydrological system of the Luxembourg sandstone region

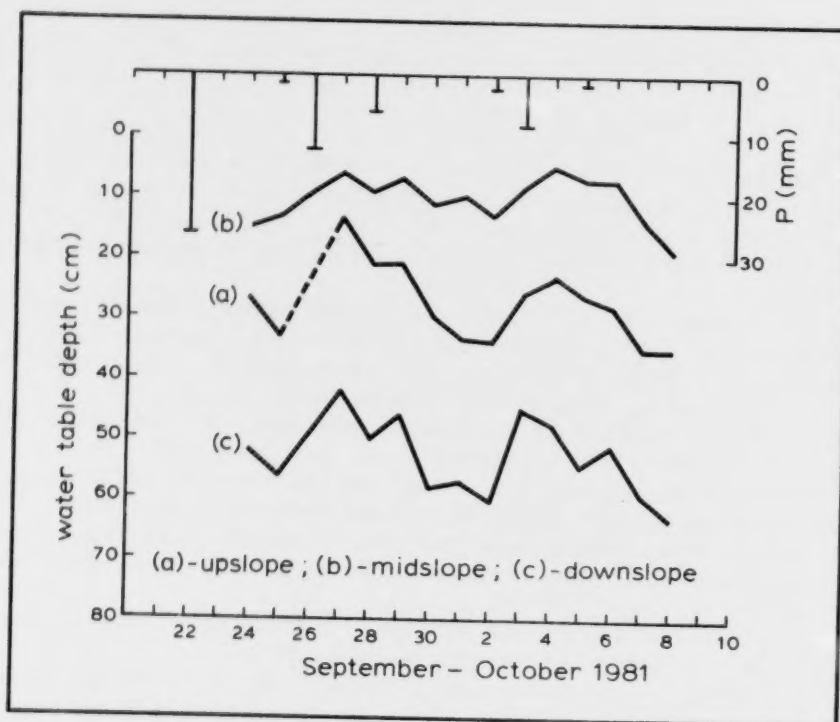


Figure 6 : Mean Dosbach piezometric head variations, sections AA', BB', CC' (24.9-8.10.81)

grams, a general distributed, physically-based model, and verification sampling. The sampling matrix approach to data regionalisation is also shown to be robust by the independent successful application of the technique to a variety of 'problems' at 'scales' which range from five to 1000 km² (see Krasovskaia, 1982; Refsgaard and Hansen, 1982; Body, 1982).

In areas where natural resource maps are available but quantitative data are scarce, this approach to regionalisation shows promise and if combined with remote sensing will certainly minimise instrumental and data collection requirements.

ACKNOWLEDGEMENTS

This project would not be possible without a team effort. The guidance and assistance given by both staff and senior students from the Department of Hydrogeology and Geographical Hydrology, Free University, are gratefully acknowledged.

REFERENCES

- Anderson, M.G. and Burt, T.P. 1978 : The role of topography in controlling throughflow generation. *Earth Surface Processes*, 3; 331-344.
- Beven, K.J. and Hornberger, G.M. 1982 : Assessing the effect of spatial pattern of precipitation in modelling streamflow hydrographs. *Water Resources Bulletin*, 18 (5); 823-829.
- Beven, K.J. and Kirkby, M.J. 1979 : A physically based, variable contributing area model of basin hydrology. *Hydrol. Sci. Bull.*, 24 (1); 43-69.

- Beven, K.J. and O'Connell, P.E. 1982 : On the role of physically-based distributed modelling in Hydrology. Institute of Hydrology, Wallingford, Report No. 81; 36 pp.
- Beven, K.J. et al. 1980 : SHE : towards a methodology for physically-based distributed forecasting in hydrology. IAHS Publication no. 129; 133-137.
- Body, D.N. 1982 : The application of results of catchment research in water resources planning and management. In, Proc. IAHS Symp. on Hydrological Research Basins and their use in Water Resources Planning, Berne, Switzerland.
- Both, M. and v.d. Sommen, J. in press : Catchment studies for the application and regionalisation of physically based hydrologic models : Luxembourg sandstone region. Dept. Hydrogeol. and Geog. Hydrol., Inst. of Earth Sci., Free University, Amsterdam, Working Paper no. 1.
- Dunne, T. 1978 : Field studies of hillslope processes. In, Hillslope hydrology, ed. M.J. Kirkby, Wiley Interscience. New York; 227-293.
- Freeze, R.A. 1980 : A stochastic-conceptual analysis of rainfall-runoff processes on a hillslope. Water Resources Research, 16(2); 391-408.
- von Hoyer, M. 1971 : Hydrogeologische und hydrochemische Untersuchungen im Luxemburger Sandstein. Publications du Service Géologique du Luxembourg vol. 21; 61 pp.
- Krasovskaia, I. 1982(a) : Hypothesis of runoff formation in small watersheds in Sweden. SMHI FoU-Notiser 19, Norrköping; 15 pp.
- Krasovskaia, I. 1982(b) : Rainfall-runoff relationships in small research basins in Sweden. In, Proc. IAHS Symp. on Hydrological Research Basins and their use in Water Resources Planning, Berne, Switzerland.
- Natural Environment Research Council (NERC). 1975 : Flood Studies Report, London, 5 vols.
- O'Loughlin, E.M. 1981 : Saturation regions in catchments and their relation to soil and topographic properties. J. Hydrology 53; 229-246.
- Pilgrim, D.H. and Bloomfield, P.H. I'A. 1980 : Problems in determining infiltration and soil parameters of runoff models. IAHS Publication no. 129; 271-277.
- Refsgaard, J.C. and Stang, O. 1981 : An integrated groundwater/surface water hydrological model. Danish Committee for Hydrology, Report SUSÅ H13; 122 pp.
- Refsgaard, J.C. and Hansen, E. 1982 : An integrated surface/subsurface catchment model. In, Proc. IAHS Symp. on Hydrological Research Basins and their use in Water Resources Planning, Berne, Switzerland.
- Seyhan, E. and Hope, A.S. 1983 : On the estimation of runoff equations and the classification of catchments in South Africa by multivariate statistical techniques. Water S.A., 8(6) (in press).
- Simmers, I., v.d. Sommen, J.J. and Both, M. 1982 : Regionalisation of hydrological data : a dynamic modelling approach. In, Proc. IAHS Symp. on Hydrological Research Basins and their use in Water Resources Planning, Berne, Switzerland.
- Tricker, A.S. 1981 : Spatial and temporal patterns of infiltration. J. Hydrology 49; 261-277.

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

SNOWMELT INDUCED URBAN RUNOFF IN NORTHERN SWEDEN

by

Lars Bengtsson
Computational Hydraulics Group
McMaster University, Hamilton, Ontario

ABSTRACT

Snowmelt and runoff data collected during 7 years at different sites in the Lule region, Sweden is summarized and analyzed. The capacity of a snowpack to hold liquid water and the effect of refreezing during cold nights are included in a degree-day approach.

The maximum observed melt flux to the base of a snowpack during one hour is 4.1 mm/h. The maximum observed daily melt is 40 mm, but the daily melt of two year return period is only 20 mm. The runoff from small study plots is (at least in practical considerations) found to be distributed over about 12 hours. Maximum observed runoff from these study plots of hard packed gravelled or grassed surfaces is about 2 mm/h. In the late phase of a snowmelt period the daily runoff is shown to closely correspond to the daily melt.

The very maximum runoff values are, however, observed during or some days after the snowmelt period. Overland flow or flow just beneath the ground surface is shown to take place.

Finally some very rough "design criteria" for snowmelt induced runoff are given.

REVIEW OF THE PROBLEM

Due to the research during the last two decades the frequency of different rain intensities over different time intervals is rather well known for populated areas in Europe and North America. From this knowledge and using physically sound runoff models the runoff due to rain from urban areas can be computed quite accurately. Computations may be made for a single rainfall event or as continuous simulation.

Only few studies of snowmelt induced runoff have been made, even if the snowmelt rate at a point has been frequently studied and analyzed in detail. The different energy fluxes, which determine the snowmelt rate is discussed

by, for example, Gray and Male (1981), who recently presented the state of the art of snow hydrology and snow management.

Before any meltwater can leave a snowpack, the snow must first be saturated above its irreducible liquid content. The meltwater, which thereafter reaches the base of the snowpack, infiltrates into the ground unless the infiltration capacity of the ground is reduced due to an asphalted surface or as a consequence of that the ground is frozen and saturated, in which case runoff can take place along the ground in the snowpack or just beneath the ground surface. The theory of percolation of meltwater through a snowpack has been developed to an advanced level through the work of Colbeck (1978). Once vertical drains develop in the snowpack, it is, however, difficult to quantify the percolation process.

There are no systematically collected data of snowmelt induced runoff or even snowmelt data at a point from urban areas reported in the literature. A group within the Swedish Board for Building Research made a state of the art report of snow management in urban areas in Sweden, Bengtsson et al. (1980). From a continuation of that work Bengtsson (1981) found that the snowmelt rate was higher in urban areas than in rural areas. He also found that in the late phase of a snowmelt period runoff took place also from normally permeable surfaces. Also the problem of infiltration in frozen soils is summarized in the snow handbook by Gray and Male.

To be able to estimate the runoff it is of course essential to know in what way the snow has been redistributed by man. Snow is piled up within the cities, but large snow masses are also transported away from the cities. The degree to which snow is transported away depends on if there is space to pile up the snow within the city, and to a large degree on how much money that has been reserved for snow management.

The above cited works and 7 years of snowmelt observations at WREL (Water Resources Engineering, Lulea, Sweden) show that at least for periods less than 6-12 hours rain intensities exceed snowmelt intensities. Still, in northern Sweden problems with damming in urban areas are much more severe during snowmelt than during heavy rain storms. After a long period of snowmelt runoff can take place from normally permeable surfaces. Rainfall during this period or just after the snow is completely gone may also cause runoff from these surfaces. When retention storages, percolation storages or infiltration surfaces are included in a storm water system, it is possible that at least some parts of the system should be designed for snowmelt conditions.

In this paper the theory of meltwater flux to the base of a snowpack is treated. Account is taken for refreezing of the liquid water of the upper part of the snowpack during cold nights. Comparison is made with measurements. Thereafter snowmelt observations from the Lulea region in northern Sweden are presented. The degree-day method, but including the capacity of the snow to hold liquid water, is used on 25 years of data for 13 areas in Sweden for estimating return periods of snowmelt intensities. Observed runoff from small areas in the Lulea region are also presented and compared with the melt rate. Rain on snow and runoff caused by rainfall just after the snowmelt period are discussed separately. Finally some rough rules of thumb are given.

All the data presented in this paper has been collected by the staff of WREL, when the author was head of the department at Water Resources Engineering, Lulea, Sweden.

SNOWMELT AND MELTWATER FLUX TO THE BASE OF A SNOWPACK

The snowmelt rate at the snow surface can be determined from an energy budget analysis. The energy balance for a snow surface is dominated by the radiation balance. A large heat flux of solar radiation and atmospheric radiation is almost completely compensated for by the reflected solar radiation and by the longwave radiation from the snow. In Fig. 1 it is shown how the measured accumulated net radiation followed the accumulated snowmelt during a snowmelt period at a site near Lulea. Although the radiation balance dominates the energy balance of a snow cover, the convective sensible heat flux between the atmosphere and the snow cover determines to a large extent how intense the snowmelt will be. In fact the degree-day method is quite accurate for determining daily snowmelt.

When consecutive snow surveys are made or snow plates are used, it is not the snowmelt at the snow surface which is measured, but how much water that leaves the snowpack or the meltflux at the base of the snowcover. The snowpack can hold liquid water. The percolation rate in dry snow is much slower than the percolation rate in snow above the irreducible liquid saturation, since rather much meltwater is needed to raise the liquid content of the dry snow to and above the irreducible value. Since the liquid content, which corresponds to a certain melt flux, does not deviate very much from the irreducible liquid content, the propagation rate of the wetting front in dry snow can as shown by Bengtsson (1982a) approximately be described by

$$C = i/\theta_i \quad (1)$$

where C = propagation rate of wetting front, i = snowmelt intensity, θ_i = irreducible liquid content (volume/volume). A typical propagation rate is of the order cm/h or even less.

The percolation rate in that part of a snowpack, which initially is at its irreducible liquid content, depends on to which extent vertical flow channels have developed and on the snowmelt intensity. Observations at WREL show typical percolation rates in the drains to be at least 0.1 but closer to 0.5 m/h.

Since a snow cover must be saturated above its irreducible liquid content before it can release any water and since intense snowmelt causes a faster percolation of meltwater than less intense snowmelt, it is often first when the surface melt is as most intense that the snow cover starts to release any water. The melt flux to the base is then close to the surface melt rate. An example from measurements at WREL is shown in Fig. 2.

From Fig. 2 it is seen that the peak melt flux to the base of the snowpack exceeds the peak value of the surface melt rate. This might be due to errors in computing the energy balance, but since the depth of the snow

cover decreases less liquid water can be held in the snowpack. Liquid water previously held in the snow is released. At constant melt rate the meltwater flux to the bottom of the snow cover is

$$v_b = i (1 + \theta_i (\rho/\rho_s)) \quad (2)$$

where v_b = meltwater flux to the base of the snowpack, ρ = density of water, ρ_s = density of snow (not including liquid water). The meltwater flux to the bottom of the snow cover can exceed the surface melt rate by as much as 15%.

During a snowmelt period the air temperature falls at least during some nights below freezing. Then the liquid water in the upper part of a snow cover refreezes. Bengtsson (1982b) has shown that the depth to which refreezing takes place can be approximately found from the implicit equation

$$\frac{\partial}{\partial t} (z_f^2) = \frac{2k (-T_a)}{a - 0.5 T_a} \quad (3)$$

where z_f = refreezing depth, t = time from when refreezing starts at the surface, k = thermal diffusivity of the snow accounting also for some convective effects due to intense winds, T_a = air temperature above freezing, and

$$a = \frac{\rho}{\rho_s} \frac{F}{c_i} \theta_i \quad (4)$$

where F = latent heat of fusion, c_i = heat capacity of ice.

For small negative values of T_a the last term of the denominator of eq. (3) can be neglected and the refreezing depth is determined as

$$z_f = C_f N^{0.5} \quad (5)$$

where C_f = refreezing coefficient, which is

$$C_f = \left(\frac{2k}{a}\right)^{0.5} \quad (6)$$

and N = number of negative degree-seconds (or more practical negative degree-hours if C_f is given in appropriate units) defined as

$$N = \int_0^t -T_a dt \quad (7)$$

Let us now return to the degree-day method, which gives a linear relation between melt rate and air temperature. From a practical point of view it is easiest to work with an equilibrium temperature of 0°C . The degree-day equation is then simply

$$m = C T_a \quad (8)$$

where M = melt rate, C = degree-day coefficient, T_a = air temperature above freezing.

The degree-day method is usually used on a daily basis or a 12-h basis but sometimes also on a monthly basis. When an equilibrium temperature of 0°C is used, the value of the degree-day coefficient depends on the solar radiation, wind conditions, the atmospheric emissivity and on the properties of the snow. For forested areas with a high canopy density it can be theoretically shown that the degree-day coefficient should be between $1.5 - 2 \text{ mm}/^{\circ}\text{C day}$, Bengtsson (1976). This theoretical value has been confirmed by many measurements, e.g. Kuusisto (1980).

For open areas degree-day coefficients determined from eq. (8) varies from region to region, area to area, and from year to year. Usually the degree-day coefficient is observed to increase in the course of the snowmelt period. This result is partly due to that it is not the surface melt but the amount of water that leaves the snow cover, which is directly or indirectly measured. But the melt rate does in fact increase. This is primarily due to the decrease of the albedo of the snow, so that more of the solar radiation is absorbed in the late phase of a snowmelt period than in the early phase. The albedo of fresh newly fallen snow is 0.8 or more. When the melt period begins it is usually 0.6 - 0.7 and drops to 0.4 during the snowmelt. In city areas the snow albedo drops rather fast to 0.2 - 0.3 as reported by Bengtsson (1981). The absorbed solar energy is almost twice as large in the late as in the early phase of a melt period.

The degree-day coefficient as determined from eq. (8) and using daily average air temperatures can well vary within an interval of $2-10 \text{ mm}/^{\circ}\text{C day}$. In Sweden the degree-day coefficient is usually higher in northern than in southern Sweden, since the melt period occurs later in spring during a higher solar intensity in northern than in southern Sweden. Since the rate of refreezing of meltwater in the snowpack is not linearly proportional to the air temperature below freezing, the high value of the coefficient is also due to the frequent large diurnal temperature variations during melt periods in northern Sweden. For open areas the degree-day coefficient can for most of Sweden be estimated to $3 \text{ mm}/^{\circ}\text{C-day}$, but in northern Sweden values around $4-6 \text{ mm}/^{\circ}\text{C-day}$ are more reasonable.

It is not obvious how a day having air temperatures above and below freezing should be treated when using the degree-day method on a daily basis. The degree-day equation gives a linear relation between melt and air temperature above freezing, whereas eq. (5) shows that the refreezing depth is proportional to the square root of the temperature below freezing. Assuming that the degree-day method is strictly correct for determining snowmelt, the average daily air temperature inserted in the degree-day equation will not give the correct melt rate, if the air temperature is below freezing during some part of the day.

Look at the following example. The average daily temperature is $+2^{\circ}\text{C}$, but during 12 h of daytime the average temperature is 4°C and during 12 h of nighttime it is -2°C . The observed snowmelt is 6 mm. When eq. (8) is applied on a daily basis, the degree-day coefficient is calculated to be $3 \text{ mm}/^{\circ}\text{C day}$.

When refreezing is taken into account and choosing $C_f = 50 \text{ mm}/(^{\circ}\text{C day})^{0.5}$, the refreezing depth is computed to 50 mm. If the irreducible liquid content is 0.04, about 2 mm of meltwater is needed for saturating the snowpack to its irreducible value. The actual surface melt is therefore the observed snowmelt 6 mm plus the above 2 mm. The total of 8 mm melts over a 12 h period, when the temperature is 4°C . The degree-day coefficient is therefore $2 \text{ mm}/^{\circ}\text{C day}$.

In the above example the degree-day coefficient was overestimated using a daily average value. If there are rather large negative temperatures during the night, the snowmelt can be rather much underestimated using the degree-day method on a daily basis and using a degree-day coefficient derived in periods of night temperatures above or only slightly below freezing.

The refreezing-degree day method suggested by Bengtsson (1982b) was from a study in Besbyn Research Basin found to be accurate on a 12 h time basis, but should be applicable also for a shorter time basis as long as the time for the water to percolate through the wetted zone of a snowpack is short. The refreezing method can be summarized as follows:

1. Determine the number of negative degree-hours over a short freezing period, for example a 24 h period.
2. Compute the refreezing depth using eq. (5) or eq. (3).
3. Compute the amount of meltwater, which is refrozen, using an a priori value of the irreducible liquid content.
- 2-3. When eq. (5) is used for computing the refreezing depth, the amount of refrozen meltwater, M_f , is

$$M_f = \theta_i C_f N^{0.5} \quad (9)$$
4. Determine the number of positive degree-hours over a short period proceeding the short period of freezing air temperatures.
5. Compute the amount of surface melt using the degree-day eq. (8).
6. Deduct the amount of refrozen meltwater, which must be remelted and held as liquid water in the snowpack before any water can leave the snowpack, from the computed surface melt to find the amount of meltwater that reaches the bottom of the snow cover.

It was seen for Fig. 2 that the melt flux to the bottom of the snowcover was distributed over about 12 hours. Even if there is melt flux also during nights, most of the meltwater reaches the ground during daytime hours. A typical runoff hydrograph observed from a snow plate at WREL is shown in Fig. 3.

OBSERVATIONS OF SNOWMELT AT A POINT

The data presented in this chapter regards the meltwater flux to the

bottom of a snow cover. Maximum melt intensities for different periods are given.

Snowmelt Intensities from Temperature and Precipitation Observations

Using 25 years of temperature and precipitation measurements the frequency of different snowmelt intensities has been computed for 13 areas in Sweden. The procedure for the computations was:

1. Update measured snow precipitation by 30% ($= S$)
2. Determine the number of positive degree-days during the melt period ($= \Sigma T$)
3. Determine the degree-day coefficient for the area and for the year as $C = S/\Sigma T$
4. Multiply the temperature, T , of the warmest day of the melt period by the degree-day coefficient. The maximum melt rate of that year is $m = C \cdot T$.
5. Distribute the snowmelt over 12 h so that $m = \text{cm}/12 \text{ h}$.
6. Update this value by 10% to account for the liquid water, which can be held by the snowpack.

Extreme value analysis has been used in analyzing the data. The computed snowmelt intensities are shown in Table 1.

The table can be summarized: Every second year the maximum daily melt is 20 mm/12 h in southern Sweden and 30 in northern Sweden.

Snowmelt Intensities from Snow Surveys

WREL is during snowmelt carrying out snow surveys every second day at a number of sites around Lulea. Since the energy balance for the snow cover is computed using measurements of radiation balance and computations of sensible and latent heat fluxes from profile measurements, eventual observation errors can be revealed fairly well, and the snowmelt over at least as short time as 2-4 days can be determined quite accurately.

The reduction of the snow cover as water equivalents is shown for 1982 for an open field and a forested area in Bensbyn Research Basin in Fig. 4 and Fig. 5. The snow cover of the forested area is reduced by 5 mm water equivalents per day during most of the melt period. The melt rate of the snow in the open field is considerably faster. All the snow disappears in the course of a week. The melt rate is somewhat more than 15 mm/day. In 1982 the snowmelt process in Lulea was, however, more even than usually with no single day of intense snowmelt.

Observations of the snowmelt have been made in Bensbyn and also at Porsön near WREL's laboratories since 1976. The observed maximum melt rates for different years are summarized in Table 2. The shown values, which are melt

TABLE 1

Recurrence interval of snowmelt runoff with intensities in mm/12 h.

Region	2 year	5 year	10 year	50 year
Karesuando	37	53	63	84
Haparanda	33	41	46	57
Stensele	22	28	32	42
Härnösand	20	30	36	51
Ostersund	28	42	51	71
Falun	24	41	52	76
Stockholm	23	35	43	59
Karlstad	21	30	35	46
Linköping	20	30	36	50
Göteborg	21	30	36	50
Visby	21	30	36	50
Växjö	20	25	30	38
Lund	22	36	46	67

TABLE 2

Maximum snowmelt runoff (mm/day) at a point having a duration of at least 3 days as calculated from snow surveys.

	1976	1977	1978	1979	1980	1981	1982
<u>Bensbyn</u> open field	13	24	40	15	15	30	15
B - small field with bushes	17	22	38	13	15	20	13
B - forested area	8	9	21	8	10	15	11
<u>Porsön</u> open field	25	24	-	20	12	-	-
P - forested area	10	10	21	11	9	-	-

fluxes to the base of the snow cover, can be assumed to occur during at least three consecutive 12 h periods.

As can be found from Table 2 the maximum observed daily melt from the open field in Bensbyn is 40 mm/day, but the median value is only 15 mm/day. For the forested area the maximum observed value is 21 mm/day and the median value 10 mm/day.

Snowmelt Intensities from Observations of Runoff from an Asphalted Surface

The runoff from a 25 m long and 8 m wide asphalted study plot having a slope of 2% has been measured with a resolution in time of 1-10 minutes during the snowmelt periods in Lulea the years 1979, 1980, 1981 and 1982. For periods of intense snowmelt over one hour or more the runoff corresponds to the melt flux to the base of the snowpack, and also to the surface melt when the reduction of the total liquid holding capacity of the snowpack is subtracted. In Table 3 maximum observed runoff over 1 h, 12 h, 24 h and maximum weekly runoff is given. The four highest independent 1-h values of runoff for the different years are given in Table 4. The corresponding 12 h runoff values are given in Table 5. Consecutive days have only been included, if the runoff ceased during the nights in between daytime hours.

The runoff over 12 h is observed to be very close to the runoff over 24 h. A very intense melt flux is about 15 mm/12 h, but in 1981 a melt exceeding 20 mm/12 h was observed at two occasions. The highest observed hourly runoff, i.e. approximately the melt flux to the base of the snow cover, is 4.1 mm. However, only at two occasions runoff of higher intensity than 3 mm/h has been observed.

Observations in Downtown Lulea

In 1980 snow surveys and runoff measurements were carried out and radiation measurements were taken in downtown Lulea. The degree-day coefficient from a small park surrounded by streets with heavy traffic was found to be 8.5 mm/°C day and for a larger park 7 mm/°C-day. The runoff was distributed over the daytime hours. During the measurement period the maximum 12 h melt was 30 mm. However, for an early warm period when no measurements were made, the 12 h melt from the two parks was using the degree-day approach estimated to 40 and 50 mm respectively.

Degree-Day Coefficient

From the snow surveys in Bensbyn Research Basin and at Porsön an average degree-day coefficient for each year and each site was determined. The calculated coefficients are shown in Table 6. The median value for "open field" is 4.8 mm/°C day and for "forest" 1.8 mm/°C day. The maximum degree-day coefficients were found for 1979. The "open field" coefficients were 8 and 12 mm/°C day, but the "forest" coefficients did not exceed the median value very much.

The refreezing-degree-day method was used on Bensbyn-data for the snowmelt period of 1981 and 1982. The degree-day coefficient was then found to be 2.8 mm/°C day for both years for the open field, and 1.4 and 1.5 for the two

TABLE 3

Maximum observed snowmelt induced runoff (mm) from a 25 m long asphalted study plot having a slope of 2%.

	1979	1980	1981	1982
1-h value	3.3	2.5	4.1	2.0
12-h value	16.9	14.6	21.3	9.6
24-h value	17.7	14.7	23.2	11.3
weekly runoff	84	75	90	50

TABLE 4

The four highest observed independent hourly runoff values due to snowmelt (mm) for different years from a 25 m long asphalted study plot having a slope of 2% in Lulea and date when the runoff was observed.

	1	2	3	4
1979	3.3 (25/4)	2.9 (28/4)	2.7 (23/4)	2.6 (24/4)
1980	2.5 (15/4)	2.1 (13/4)	1.9 (12/4)	1.9 (16/4)
1981	4.1 (18/4)	3.0 (17/4)	2.7 (17/4)	2.1 (19/4)
1982	2.0 (17/4)	1.8 (1/4)	1.4 (26/3)	1.4 (21/4)

TABLE 5

The four highest observed independent 12-h runoff values (mm) for different years from a 25 m long asphalted study plot in Lulea and date when the runoff was observed.

	1	2	3	4
1979	16.9 (23/4)	16.0 (28/4)	15.6 (25/4)	-
1980	14.6 (15/4)	12.4 (14/4)	10.6 (13/4)	10.2 (7/4)
1981	22.3 (11/4)	21.4 (18/4)	10.1 (14/4)	8.9 (15/4)
1982	9.6 (1/4)	8.5 (17/4)	8.0 (26/3)	7.4 (16/4)

TABLE 6

Degree-day coefficient (mm/°C day) estimated from snow surveys for entire melt periods using 0°C equilibrium temperature.

	1975	1976	1977	1978	1979	1981	1981	1982
<u>Bensbyn</u> open field	-	2.8	5.9	7	8	4.8	3.1	3.9
B - small field with bushes	-	2.3	3.7	6	6	4.8	-	-
B - forested area	-	1.8	2.2	2.1	2.2	1.8	1.5	1.6
<u>Porsön</u> open field	4.0	2.9	6.5	10	12	3.7	-	-
P - forested area	2.1	1.8	2.2	2.2	2.5	1.8	-	-

years for the forested area.

When refreezing is accounted for the degree-day coefficient has more physical significance than when it is not accounted for. The coefficient is reduced and is more constant, which enables better forecasts.

The runoff data for 1981 from the asphalted study plot at WREL was used by Westerström (1982) for determining the degree-day coefficient using different temperature index. He separated "early" and "late" melt. When the daily mean temperature was used as the temperature index the degree-day coefficient for early melt was found as $1.7 \text{ mm/}^{\circ}\text{C day}$ and for late melt (after about 25 accumulated degree-days) as 6.5. When only the part of the day having air temperatures above freezing was included in the temperature index (which then corresponds to for example number of degree hours above freezing), the degree-day coefficients for early and late melt were found to be 1.5 and $4.4 \text{ mm/}^{\circ}\text{C-day}$ respectively. The "late" melt period using this temperature index could not be considered to have started until after about 45 accumulated degree-days ($^{\circ}\text{C-days}$).

SNOWMELT INDUCED RUNOFF

Infiltration

When a snowpack has been saturated above its irreducible liquid content and meltwater reaches the base of the snow cover, the melt-water can, depending on the conditions of the soil, either infiltrate into the soil or run along the ground. The soil beneath a melting snowpack is usually frozen. Even if the ground has not been asphalted, the infiltration capacity of the soil is reduced. In the early phase of a melt period all the meltwater can usually infiltrate into the soil. Then the soil moisture is gradually increased. Also some of the meltwater refreezes, when it penetrates into the frozen soil. The infiltration capacity of the soil is reduced. In the late phase of a long melt period the infiltration may cease completely. Since the total melt amounts to large quantities, it may also be that the groundwater level rises almost to the ground surface.

The theory of infiltration is not treated in this paper. It is only noted that there is a relation between negative soil temperature and the amount of liquid water in the frozen soil. If the moisture content of the soil is high when the soil freezes in the autumn, and if the ground is cooled down and very frozen before it is covered by an insulating snow cover, the ice content of the soil will be large and the infiltration capacity reduced very much. Then only a small amount of meltwater can be introduced into the soil before runoff takes place along the surface.

A measure of the infiltration capacity of the frozen soil can be obtained by measuring how much the water equivalents of a snow cover is reduced before any runoff is observed in nearby small perennial streams. Such observations have been made in Bensbyn Research Basin. The observed reduction of the snow cover for consecutive years is shown in Table 7. After the runoff has started the daily runoff corresponds almost to the daily melt found from the snow large open field, which constitutes 30% of the 1.6 km^2 catchment, is meadow

TABLE 7

Reduction of the water equivalents of the snow cover + rain precipitation (mm) during different snowmelt periods up to the time when runoff was first observed.

	1977	1978	1979	1980	1981	1982
Bensbyn - open field	70	90	90	-	100	10
Bensbyn - forested areas	70	50	-	-	100	10

grass. There is a sandy layer of some 10-30 cm above a silty soil.

Runoff from Bensbyn

The Bensbyn Research Basin can be divided into two parts. The upper part, about 1.1 km², of the catchment is covered with dense coniferous forest. The lower part, about 0.5 km², is flat and is mainly a large meadow. Across the meadow there are two small ditches. Two small rivulets, which are dry most of the year, meet at the edge of the meadow. The discharge is measured a few hundred metres below the confluence.

Maximum observed runoff from Bensbyn for different years is given in Table 8. For daily runoff only one of consecutive days of high discharge has been included.

From Table 8 it is seen that the maximum daily runoff every year during the period 1977-1982 has been at least 6 mm. The three highest daily runoff values without contribution from rainfall are 18, 12 and 11 mm. The corresponding highest 1-h runoff values are 0.75, 0.54 and 0.53 mm/h. The largest weekly runoff corresponds to 9 mm/day.

The very highest runoff from Bensbyn has, however, been observed as a consequence of rainfall just after the snowmelt period in 1982. A daily runoff of 26 mm and an hourly peak value of 1.3 mm/h were observed.

The runoff given in Table 8 is referred to the area of the entire catchment. However, the snowmelt starts earlier from the open area than from the forest. Many years the snow in the open field has disappeared completely before runoff commences from the forested areas. The snowmelt induced runoff from the separate areas is therefore higher than the average value for the whole catchment as given in Table 8.

During 1978, 1980 and 1981 the maximum discharge in the small stream was not observed until the open field was free from snow. For these three years it was possible to separate the runoff from the forest and the meadow. The maximum daily runoff for the two areas and the three years is given in Table 9.

TABLE 8

The three highest observed runoff values (mm) for different time intervals from Bensbyn Research Basin and date when the runoff was observed.

	1977	1978	1979	1980	1981	1982
1-h value	0.80 ^x (16/5)	0.34 (16/5)	-	0.32 (23/4)	0.54 (13/5)	1.28 ^z (29/5)
	0.75 (5/5)	0.32 (17/5)	-	0.29 (1/5)	0.53 (12/5)	0.39 ⁱ (9/5)
	0.58 ^x (13/4)	0.30 (18/5)	-	0.28 (4/5)	0.53 (14/5)	0.34 (25/4)
daily runoff	17.8 (5/5)	7.3 (17/5)	12.2 (4/5)	6.0 (1/5)	11.1 (14/5)	26.0 ^z (29/5)
	16.2 ^x (16/5)	6.0 (19/5)	9.7 (2/5)	5.6 (4/5)	10.3 (12/5)	7.8 ⁱ (9/5)
	11.9 ^x (13/5)	4.4 (15/5)	9.7 (30/4)	5.1 (23/4)	8.0 (10/5)	6.0 (25/4)
weekly runoff	8.1 (13-19/5)	40 (15-21/5)	63 (30/4-6/5)	36 (30/4-1/6)	68 (11-17/5)	43 (4-10/5)

x) during the period 12-14 May the rain precipitation was 12 mm.

z) during the previous day the rain precipitation was 60 mm.

i) during the previous day the rain precipitation was 9 mm.

TABLE 9

Maximum snowmelt induced daily runoff (mm) from the forested area and the open field of the Bensbyn Research Basin for the years 1978, 1980 and 1981.

	1978	1980	1981
Bensbyn - open field	10.4	8.6	19.9
Bensbyn - forested area	10.8	5.7	23.0

It should be noted that the design runoff values for stream flow generated from farming areas suggested in Swedish handbooks is 13 mm/day.

Runoff from WREL Study Plots

In a previous chapter runoff data from an asphalted study plot at WREL was presented. Now this data will be compared with the runoff from two study plots with grassed and gravelled surfaces. The area of these two study plots is for the asphalted surface 8 x 25 m² and the slope is 2%. The three plots are next to each other. The grassed surface is shaded by a house towards the evening, so the melt rate at the grassed surface may in late afternoon be reduced compared to the melt rate at the asphalted surface.

The most intense observed runoff for different years is given in Table 10 and as runoff over 12 h in Table 11. All of these runoff values were exclusively due to snowmelt.

The maximum weekly runoff varied between values corresponding to 7-14 mm/day for the asphalted surface, 3-11 for the gravelled surface and 1-8 for the grassed surface. In 1980 the maximum weekly runoff corresponded to 11 mm/day from the asphalted as well as for the gravelled surface. No data exists for that year for the grassed surface, but usually the runoff from the surface covered with grass is very close to the runoff from the surface covered with gravel.

The runoff from the surface covered with gravel may be as high as from the asphalted surface as was found in 1980, when all the three daily maximum runoff values were equal. All the meltwater ran along the ground. Usually, however, the runoff from the grassed and gravelled surfaces is about half the melt value found as the runoff from the asphalted surface. When plotting accumulated runoff versus accumulated degree-days Westerström (1982) found the ("runoff"-) degree-day coefficient for grass and gravel to be about half of the degree-day coefficient for snowmelt.

Runoff Observations in Downtown Lulea

During the snowmelt of 1980 some few measurements of the runoff from a 6500 m² peak, where the snow had been removed from the gravelled paths, were made. The maximum observed runoff corresponded to 2.3 mm/h, when the area of the gravelled paths was excluded. This peak value is in agreement with the data from the WREL study plots.

Rain During the Snowmelt Period

Rain on snow has only a minor effect on the melting process, but it has the effect that it increases the degree of liquid saturation of a snow cover. If the snow is wet when the rain starts to fall, all the rain precipitation reaches the bottom of the snow cover. In the late phase of a snowmelt period all the rain precipitation may run off as overland flow.

When a thick snow cover is completely gone, the infiltration capacity of the soil is still very much reduced. Most of the rain precipitation on this

TABLE 10

The three yearly highest observed hourly runoff values (mm) from 25 long, 2% sloping study plots, WREL. Date of observation is given within brackets.

	1979	1980	1981	1982
asphalt	3.3 (25/4)	2.5 (15/4)	4.1 (18/4)	2.0 (17/4)
	2.9 (28/4)	2.1 (13/4)	3.0 (11/4)	1.8 (1/4)
	2.7 (23/4)	1.9 (12/4)	2.7 (17/4)	1.4 (26/3)
gravel	2.4 (28/4)	2.2 (15/4)	1.9 (18/4)	1.2 (17/4)
	1.6 (25/4)	2.1 (13/4)	2.6 (20/4)	0.8 (18/4)
	0.9 (29/4)	2.0 (12/4)	1.6 (11/4)	0.7 (16/4)
grass	-	-	1.8 (18/4)	0.2 (17/4)
	-	-	1.6 (20/4)	0.1 (23/4)
	-	-	1.2 (11/4)	0.1 (18/4)

TABLE 11

The three yearly highest observed 12-h runoff values (mm) from 25 m long, 2% sloping study plots, WREL, Lulea. Date of observation is given within brackets.

	1979	1981	1981	1982
asphalt	16.9 (23/4)	14.6 (15/4)	22.3 (11/4)	9.6 (1/4)
	16.0 (28/4)	12.4 (14/4)	21.4 (18/4)	8.5 (17/4)
	15.6 (25/4)	10.6 (13/4)	10.1 (14/4)	8.0 (26/3)
gravel	12.7 (28/4)	14.2 (15/4)	11.0 (18/4)	4.4 (16/4)
	8.2 (25/4)	12.4 (14/4)	10.1 (11/4)	37 (18/4)
	4.9 (29/4)	10.6 (13/4)	5.8 (19/4)	3.4 (16/4)
grass	-	-	11.6 (18/4)	1.0 (17/4)
	-	-	8.5 (11/4)	0.7 (23/4)
	-	-	6.7 (20/4)	0.5 (18/4)

wet soil, which still is frozen at some depth below the ground, may except for evaporation losses run off as overland flow.

The runoff from the Bensbyn Research Basin was observed to be high during the period 12-17 May 1977. By that time the open field was snow free. From energy balance computations based on measurements of radiation balance, wind-, temperature-, and humidity profiles and controlled by snow surveys, the total snowmelt from the forested areas was for the period calculated to be 47 mm with reference to the area of the whole catchment. The rain precipitation was 12 mm. The observed total runoff of 56 mm almost matched the sum of the total melt and the rain precipitation.

The peak runoff from Bensbyn during the snowmelt period of 1982 was observed on 9 May. At that time the open field was free from snow. The melt rate in the forest was during 8-10 May about 7-8 mm/day, which with the total area as reference corresponds to 5 mm/day. The rain precipitation was 6 mm on 8 May and 3 mm the day after. The mean runoff of 9-10 May was almost 7 mm/day, which should be compared to the mean melt rate and precipitation, 9.5 mm/day. Most of the rain ran off rather fast.

The highest discharge ever in the small Bensbyn stream was recorded on 29 May 1982. The open field had been snow free for one month, and the last snow in the forest had melted two weeks earlier. During the night between 28 and 29 May there fell 61 mm of rain. At 9 o'clock in the evening of 28 May the low discharge of 21 l/sec in the stream began to increase. A peak value of 570 l/sec was observed at 4 o'clock the following afternoon. By noon 31 May the discharge had decreased to 80 l/sec. The total runoff during the two days 29-30 May only corresponded to 38 mm.

From the above rather brief analysis of the combined snowmelt-rain runoff it can be concluded that rain on snow or rain directly on frozen soils can give rise to fast runoff from normally permeable surfaces.

SUMMARY OF THE MEASUREMENTS

The melt rate of 2 year return period was from precipitation and temperature observations estimated to 20 mm/12 h for open areas in southern Sweden and to 30 in northern Sweden. The values for 10 year recurrence interval were 30 mm for southern and 40 for northern Sweden.

From snow surveys over 7 years around Lulea the median value of maximum daily melt was found to be 15 mm for open fields and 10 mm for dense forest. The corresponding maximum observed values were 40 and 21 mm/day, respectively.

The degree-day coefficient for forested areas is about 4 mm/°C-day. The degree-day coefficient for open areas may vary between 2-10 mm/°C-day. By accounting for refreezing of the liquid water held by the snowpack and applying a refreezing-degree-day approach, a rather constant degree-day coefficient, 3-4 mm/°C-day, can be used, which enables more accurate forecasts.

The hourly maximum runoff observed from a small impermeable study plot at

WREL is 4.1 mm/h. However, only twice a melt flux exceeding 3 mm/h has been observed. Most of the melt flux reaches the bottom of a snow cover over a 12 h period. The 12 h runoff is very close to the total daily runoff. The maximum observed runoff over 12 h is 22 mm.

In the late phase of a melt period the infiltration capacity of normally permeable soils may be almost completely reduced. Overland flow can take place also from surfaces covered with grass or gravel. In the Bensbyn Research Basin runoff from an open field is usually not observed until on the average 70 mm of meltwater has infiltrated into the soil, and runoff from the forest not until about 40 mm has infiltrated. Thereafter the runoff corresponds almost to the melt rate. The values given above vary from year to year and depend on the soil conditions by the time of the first snow cover in early winter.

The maximum observed daily runoff from the open field in the Bensbyn Research Basin corresponds to 23 mm and that from the forested area to 16 mm. From an analysis of the diurnal discharge fluctuations in the Bensbyn stream Bengtsson (1982 c) suggests that half of the daily runoff is contributed to groundwater.

The measurements of runoff from the WREL study plots show that in the late phase of a melt period the runoff from packed grassed and gravelled surfaces may be very close to the runoff from asphalted surfaces.

The maximum observed runoff from the study plots covered with grass and gravel is 2.4 mm/h, which is in agreement with measurements from a city park in downtown Lulea. The maximum observed daily runoff from the study plot with gravel is 14 mm, but almost all the meltwater ran off during a 12 h period. The total weekly runoff for that period was 80 mm.

When rain falls on snow or just after a snowmelt period almost all of the rain may run off as overland flow also on surfaces covered with grass or gravel.

SOME RULES OF THUMB FOR ENGINEERS

There are no design criteria for snowmelt. From the experience gained at WREL the following very rough rules of thumb are suggested for Swedish conditions.

A design melt rate of 1 year return period is for open areas in Sweden 20 mm/12 h and for a shorter interval 3 mm/h. For urban areas a value of at least 30 mm/12 h is suggested. For forested areas the value should be 10 mm/12 h. Rain during the snowmelt should be added to the melt. The melt is linearly distributed over 12 h.

If there is much rain in the autumn and if the soil temperature is low before a snow cover is formed, overland flow can be expected to take place from all surfaces in the late snowmelt period. If the soil is packed hard, the runoff may correspond to the melt rate.

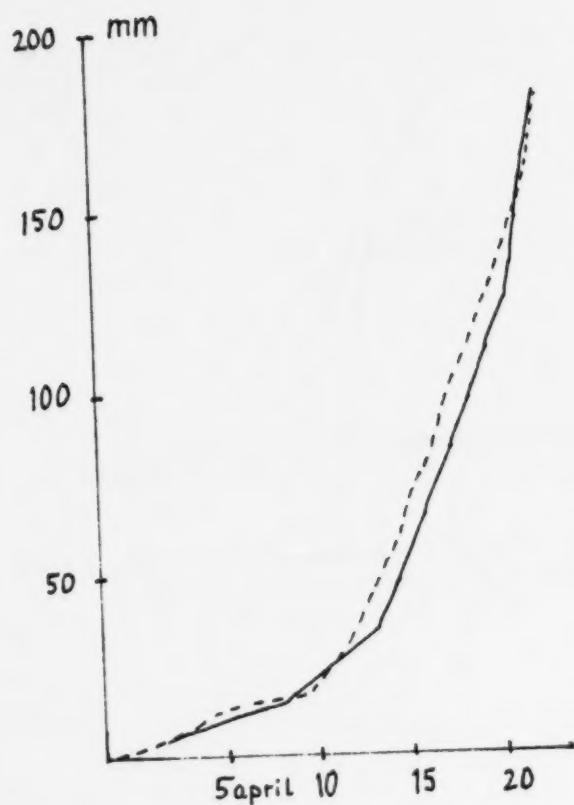


Figure 1 Observed accumulated snowmelt 1976 (solid line) from the open field in Bensbyn Research Basin and the measured radiation balance as corresponding to melt (dashed line).

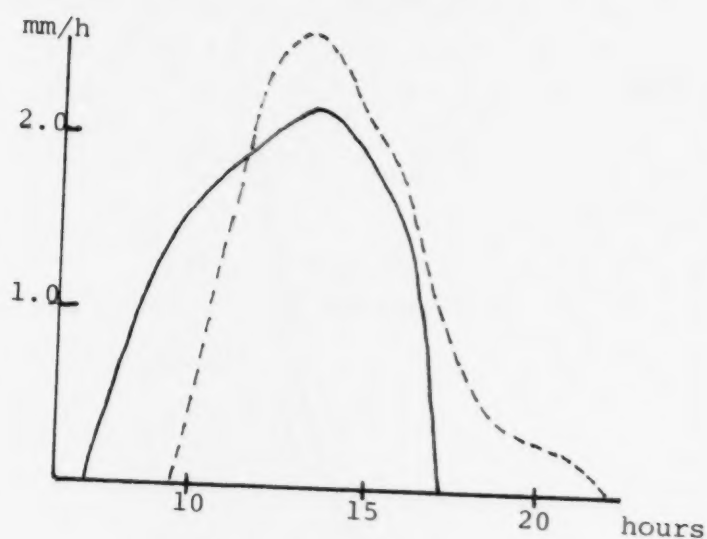


Figure 2 Snowmelt intensity (solid line) from measured at WREL, 24 April 1979 and observed runoff from a snow plate (dashed line).

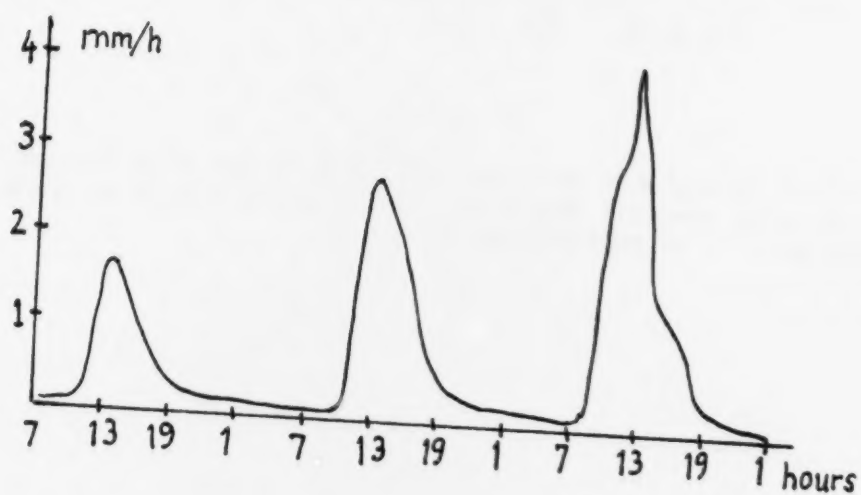


Figure 3 Runoff 16-18 April 1981 from the short asphalted surface study plot at WREL Water Resources Engineering, Lulea.

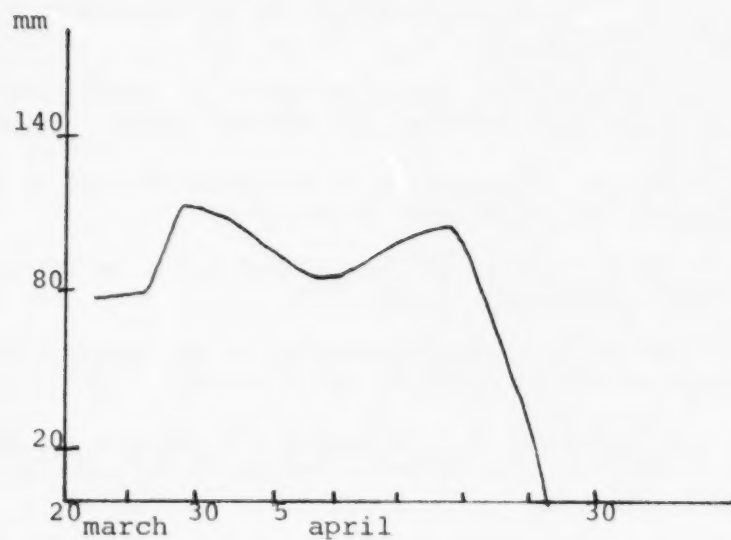


Figure 5 Water equivalents of the snow cover in the open field in Bensbyn Research Basin 1982.

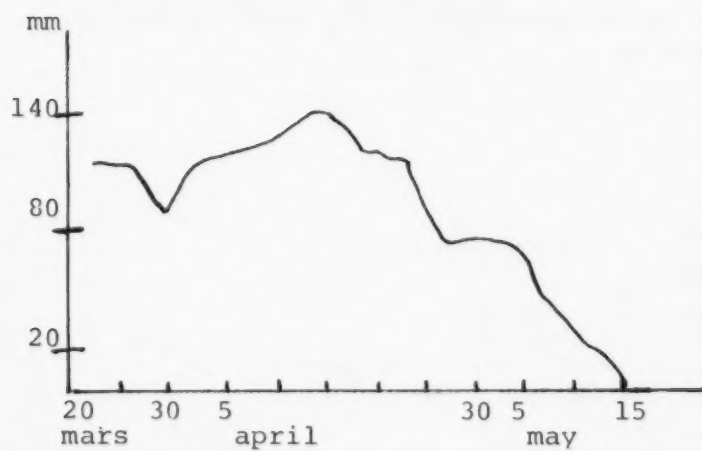


Figure 6 Water equivalents of the snow cover in the forest in Bensbyn Research Basin 1982.

REFERENCES

- Bengtsson, L. (1976), "Snowmelt estimated for energy budget studies", *Nordic Hydrology* 7, pp. 3-18.
- Bengtsson, L. (1981), "Snowmelt generated runoff in urban areas", *Proc. Second Int. Conf. Urban Storm Drainage*, pp. 444-451, Urbana, Illinois.
- Bengtsson, L. (1982 a), "Percolation of meltwater through a snowpack", *Cold Regions Science and Technology*, 6, pp. 73-81.
- Bengtsson, L. (1982 b), "The importance of refreezing on the diurnal snowmelt cycle", *Nordic Hydrology*, 13, pp. 1-12.
- Bengtsson, L. (1982 c), "Ground and meltwater in the snowmelt induced runoff", *Hydrological Sciences Journal* 27, pp. 147-158.
- Bengtsson, L., A. Johnsson, P.A. Malmquist, E. Särner, J. Hallgren (1980), "Snow management in urban areas", *Swedish Council of Building Research, Report R27*, 1980.
- Colbeck, S.C. (1978), "The physical aspects of water flow through snow", *Adv. Hydrosol.* 11, pp. 165-206.
- Gray, D.M. and D.H. Male (1981), *Handbook of Snow*, Pergamon Press, Toronto.
- Kuusisto, E. (1980), "On the values and variability of degree-day melting factor in Finland", *Nordic Hydrology* 11, pp. 235-242.
- Westerström, G. (1982), "Estimating snowcover runoff by the degree-day approach", *Nordic Hydrologic Conf.*, Forde, Norway, WREL, Ser. A, No. 107, TULEA, 1982: 29.

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

AN ADVANCEMENT IN HYDRAULIC MODELING OF POROUS PAVEMENT FACILITIES

Gary F. Goforth
Espey, Huston & Associates, Inc.
Austin, Texas

ACKNOWLEDGEMENTS

The study addressed in this paper was part of a comprehensive investigation of operational characteristics of porous and conventional paving systems conducted by Espey, Huston & Associates, Inc. and the City of Austin, Texas. The project was executed in fulfillment of Grant No. R806338-01-2 with the City of Austin, under the sponsorship of the U.S. Environmental Protection Agency. A report documenting the entire project will be completed during the summer of 1983.

INTRODUCTION

The liabilities of urban development to indigenous water resources are generally accepted to be manifested in stormwater runoff peaks and event volumes of greater magnitude than in the predeveloped state, often occurring in association with a degradation of receiving water quality. The increase in impervious areas such as roofs, streets, and parking lots in urban areas reduces the infiltration capacity of urban watersheds and produces a corresponding increase in runoff rates and volumes. Stormwater runoff from developed areas has been recognized as a source of contaminant loading to surface and ground water bodies. Impervious areas generally have limited assimilative properties and in some cases tend to yield contaminants that are not amenable to control and removal using standard treatment procedures(1). Stormwater flows transport contaminants which have accumulated on the watershed during dry weather, however, the total mass transported is a function of the contaminant accumulation rate, the number of antecedent dry days, the intensity of the rainfall, the velocity and volume of surface flow and other physical properties of the catchment. Heavy metals, exhaust products, oils and other hydrocarbons from automobiles and machinery, suspended solids from dust and dirt accumulation and airborne pollutants washed out during precipitation events are typical contaminants present in urban stormwater runoff(4).

Stormwater management generally consists of collecting and transporting overland runoff in a conveyance system of storm sewers and possibly channels which are tributary to a nearby stream or lake. Although local flooding problems may be solved by this system, the shorter time of concentration and higher peak flows which are generated may tend to create more severe flood problems downstream. The increase in flow velocities in the improved channels creates a high erosion and scour potential, thus exacerbating the problem of pollutant transport to receiving bodies of water.

POROUS PAVEMENTS

A porous pavement facility is an innovative solution to the problem of stormwater drainage from parking and other low density traffic areas in the urban landscape. This type of pavement uses the natural infiltration capacity of the soil to absorb rainfall and local runoff after accumulation in a porous base, consisting of sand or large diameter open graded gravel. If infiltration into the soil is undesirable or not practical, lateral drainage to a sump or channel may be provided. Porous pavement systems can be designed to minimize changes in the runoff quantity and quality characteristics of a watershed during and after development. A cross-section of a typical porous pavement facility is presented in Fig. 1.

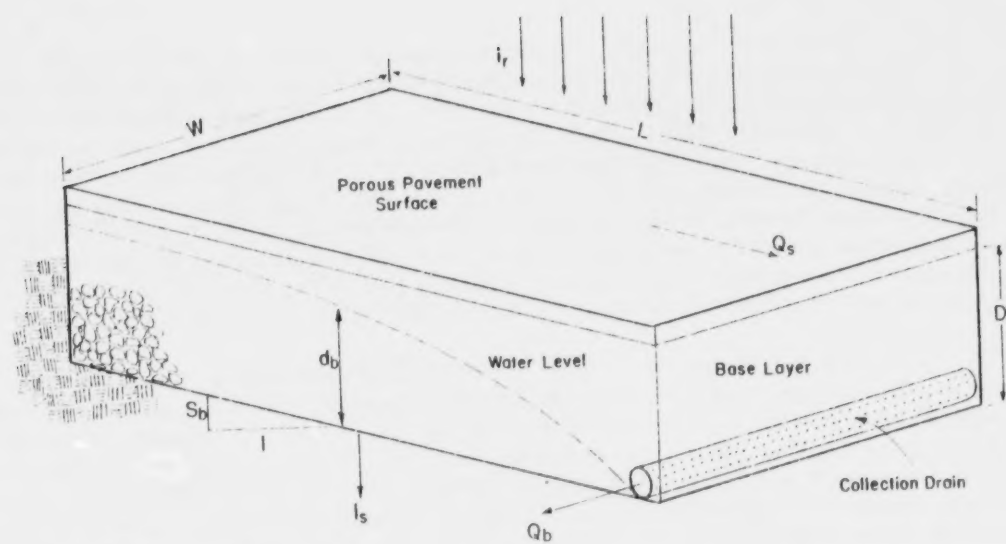
In regular applications for highway and airport runway construction, a commonly used porous pavement surface has been referred to as plant mix seal coat, open graded mix, gap graded mix, popcorn mix, or porous friction course(2). This material consists of an open graded asphalt and concrete mixture with a high percentage by weight of aggregate larger than a number four seive, laid to a thickness of $\frac{1}{2}$ to 1 inch. The resultant paving has a coarse surface texture and a high void ratio resulting in temporary storage of surface water while maintaining the coefficient of friction between a vehicle tire and pavement at values comparable to the coefficient under dry conditions. The open graded asphalt mixture underlain by a gravel base course with appreciable storage capacity is the most frequently used type of porous pavement. The whole system may be isolated from the natural ground by an impermeable membrane such as a polyethylene liner, in which case some type of artificial drainage would be needed; or, the porous pavement system may be allowed to drain into the natural ground at all points of contact. The latter arrangement does not preclude the use of artificial drains, as in the case of highly impervious natural ground. Where storage is provided, flow control devices can be incorporated to regulate the release rate, e.g., to prevent discharge during a predetermined period after a storm event.

By design, several inches of rainfall and runoff can be stored within a porous pavement system prior to discharge. The pavement can be designed to retain all of the runoff with no drainage from the site, to retain a sufficient volume of runoff to reduce the after-development hydrologic conditions to predevelopment conditions, or to delay runoff from the site, thus attenuating peak discharges and reducing the impact of associated pollutant transport. Any combination of these properties can be incorporated into an overall project drainage design to satisfy municipal or watershed drainage management criteria.

Pollutant removal mechanisms in a porous pavement system have not been fully documented. The relatively slow hydrodynamics may allow some settling of suspended matter. Adsorption to and absorption in the base media may also be realized. Although transport of soluble constituents into the ground via infiltration removes them from the porous pavement system, subsequent groundwater transport to receiving waters may result.

SIMULATION OF STORMWATER HYDRAULICS

Prediction of hydraulic characteristics is a valuable tool for assessing the performance of stormwater runoff control strategies. Stormwater hydraulic



- L length of pavement
- W width of pavement
- D depth of base layer
- S_b slope of base layer
- i_r rainfall intensity
- i_s infiltration rate
- d_b depth of water in base layer
- Q_s surface discharge
- Q_b collection drain discharge

Fig. 1.. CROSS SECTION OF TYPICAL POROUS PAVEMENT FACILITY

characteristics of porous and nonporous pavement study sites were evaluated using a revised version of the computer model PORPAV, initially developed for incorporation into the EPA Storm Water Management Model (SWMM)(3). PORPAV is a two-dimensional dynamic water budget analysis of a pavement facility. The utilization of PORPAV allows a comprehensive analysis of flow and storage in porous and nonporous pavement facilities, facilitating comparisons of the hydraulic response of alternative pavement designs. The computational scheme of PORPAV is described below.

The rate of inflow to the pavement facility from rainfall and, if present, any contributing area is compared to the permeability of the porous pavement for each time interval. For porous systems in general, the permeability is much greater than the inflow rate and all of the water moves into the pavement control volume. For nonporous pavements the permeability is generally less than the inflow rate and limited portion of the inflow moves into the pavement. The excess is stored on the surface of the pavement for subsequent computation of surface runoff from the facility.

The inflow into the pavement control volume is added to the existing storage and then compared to the permeability of the base. If the base permeability is greater than the stored volume in the pavement, all of the flow is transferred into the base control volume. This is true for most porous pavement systems operating according to design. In those instances where the base permeability is less than the inflow volume, the inflow into the base is computed as the vertical seepage into the base, at a rate limited by the smaller of the pavement or base permeabilities. The lateral outflow from the pavement top layer volume is assumed to be negligible as compared to the vertical flux. The difference between the flow into the pavement and the transport to the base layer is stored in the pavement control volume.

The inflow to the base control volume is added to the existing base volume. The revised PORPAV includes a routing procedure to account for the vertical transport of water within the layers, simulating the vertical movement of the wetting front as it passes through the pavement system. In essence, volumes of water, defined by the permeability of the layer and the length of the computational time step, are routed through the depth of the layer. PORPAV allows the option of utilizing single or multiple collection drains for discharge from the base layer. An expression for estimating the horizontal discharge from the porous pavement base was developed to reflect the nonsteady flow regime in the porous media. Darcy's Law was employed as the governing flow equation(5),

$$Q_b = K_b A dh/dx$$

where Q_b is the average horizontal discharge;
 K_b is the permeability of the base media;
 A is the cross-sectional area of flow; and,
 dh/dx is the energy gradient.

The energy gradient was approximated by

$$dh/dx = H/L$$

where H is the total elevation potential, equal to $d_b + LS_b$;
 d_b is the depth of water in the base layer;

L is the normal length of the base layer; and,
 S_b is the slope of the base layer.

The cross-sectional area of flow was approximated as

$$A = wd/2$$

where w is the width of the layer.

This yields

$$Q_b = (K_b w S_b) d_b/2 + (K_b w) d_b^2/2L$$

or on a unit area basis,

$$q_b = c_1 d_b + c_2 d_b^2$$

where $c_1 = K_b S_b/2L$; and,

$$c_2 = K_b/2L^2$$

When there is no impermeable seal present to restrict flow, some horizontal discharge will occur to the adjacent soil. However, this horizontal flux is generally negligible when compared to the vertical component leaving the layer via infiltration because of the much smaller cross-sectional area of flow. Also, the moisture content of the surrounding soil increases during the storm event, thereby reducing the hydraulic energy gradient between the porous media and the soil. In a narrow, high-wall trench without an impermeable seal or a drain pipe, the horizontal flux to the soil during the initial period of the storm, before the hydraulic gradient between the base and the soil diminishes, may be of the same order of magnitude as the infiltration flux. By neglecting the horizontal flux when there is no impervious seal present this assumption represents a conservative case with regard to the effective storage of the porous pavement system.

PORPAV incorporates Horton's equation to describe the variable rate of infiltration during and subsequent to a precipitation event. This was expressed as

$$i_s = i_f + (i_o - i_f) e^{-kt}$$

where i_s is the infiltration rate at time t ;
 i_f is the infiltration capacity (minimum rate) of the soil;
 i_o is the initial infiltration rate;
 k is the first-order decay coefficient; and,
 t is the elapsed time.

The initial infiltration rate is dependent on the initial moisture condition of the soil. The difference between the initial and minimum infiltration rates can result in significantly greater vertical transport calculated during the storm event. If a constant infiltration rate is preferred, the initial rate may be replaced by the infiltration capacity (minimum rate) of the soil. If the bottom is sealed with an impermeable membrane, no flow is discharged to the natural ground. The difference

between the inflow to the base layer and the outflows (vertical and lateral) from the base is stored in the base control volume.

All stored volumes within each layer are compared to maximum void volumes. If the storage volume in the base is exceeded, the excess is stored in the pavement; if the storage volume in the pavement is exceeded, the excess is added to the surface storage on the pavement, if any exists. Surface runoff is then computed as broad channel flow from the pavement using Manning's equation.

Provisions were added to PORPAV to calculate the theoretical detention time provided by a pavement facility. This duration was calculated as the elapsed time between the center of mass of the inflow hydrograph and the center of mass of the discharge hydrograph. Additionally, average and cumulative inflow, peak and cumulative discharge and other discharge hydrograph characteristics are compiled for each simulation.

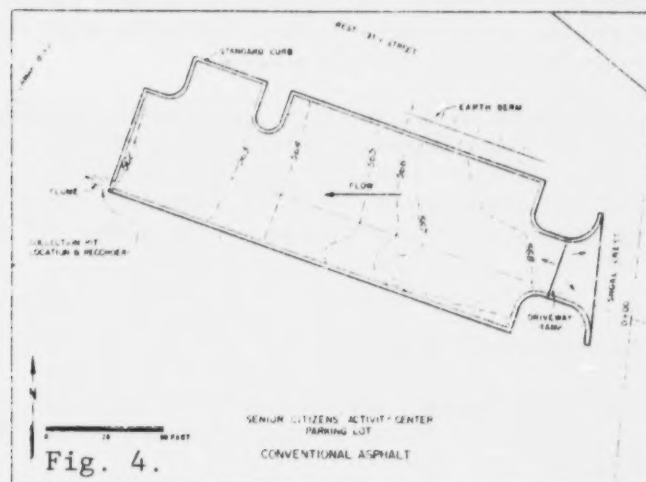
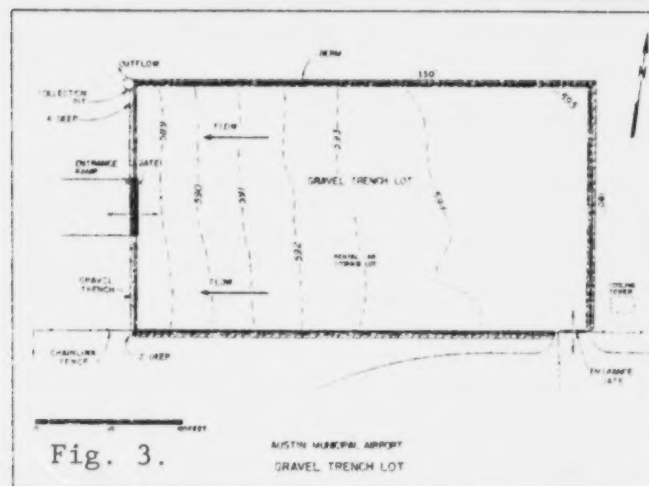
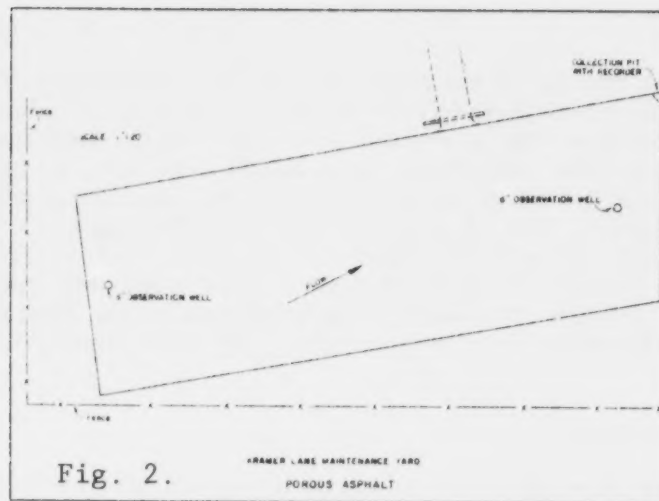
DESCRIPTION OF THE MONITORING SURVEYS

An extensive monitoring program was initiated to document hydraulic characteristics of several pavement systems. A monitoring network of three parking lots was selected, representing a variety of pavement surfaces. The following text discusses the physical characteristics and sampling procedures for each of the study lots.

Porous Asphalt Lot

A plan drawing of the porous asphalt study lot is presented in Fig. 2. The porous asphalt lot consisted of three layers of stone and asphalt constructed on an impervious limestone bedrock base. The lowest layer was made up of a stone base course with rocks ranging from 1.5 to 2.5 in (3.0-6.5 cm) in diameter. This base ranged in depth from approximately 4 in (10 cm) on the upslope end to 42 in (107 cm) on the low end and provided a void space of approximately 40 percent of its volume for water retention. The second layer averaged 2 in (5 cm) in depth and consisted of a stone top course (filter course) with material ranging from 0.4 to 0.6 in (1.0-1.5 cm) in diameter. This intermediate layer was selected to provide a uniform surface for the application of the porous asphalt. The final layer consisted of 2.5-in (6.4 cm) of porous asphalt mix with 5.5 to 6.0 percent asphalt content.

The original design called for runoff to be monitored within a collection basin located at the downslope corner of the lot. A 6-mil polyethylene impermeable liner was installed along the above-grade downslope width and side to prevent seepage. However, this seal leaked and the base discharge did not flow into the monitoring barrel. Small trenches and berms were constructed along the periphery of the lot to ensure all runoff was captured. However, these trenches were not lined and, consequently, some percolation and erosion inevitably occurred. A 90-degree V-notch weir was installed below the lot in order to measure the discharge rates. Visual stage readings at this weir by the field crew were used as a basis for the runoff calculations.



Gravel Trench Lot

A plan drawing of the gravel trench lot is presented in Fig. 3. The study area consisted of a conventional asphalt lot with a 4-ft (1.2-m) wide and 3-ft (0.9-m) deep drainage trench at the downslope end. The trench was lined with a 6-mil polyethylene impermeable membrane and filled with 1.5 to 2.5 in (3.8-6.4 cm) diameter crushed stone, cleaned and washed. This base was topped with approximately 1 ft (30 cm) of smaller than 1-inch (2.5-cm) diameter gravel. The trench was subsequently flushed with several volumes of water to rinse out construction fines. Stormwater flows within the drainage trench were monitored within a 55-gallon (208-l) barrel. A 4-in (10-cm) diameter pipe was used as the discharge control. All discharges were calculated based on readings of the water level within the barrel by the field crew.

Conventional Asphalt Lot

A plan drawing of the conventional asphalt lot monitored in the Austin study is presented in Fig. 4. Runoff discharge estimates were estimated from water levels in a 3 x 3 x 1.5 ft (0.9 x 0.9 x 0.5 m) 90-degree V-notch box weir.

SPRINKLER-GENERATED RUNOFF EVENTS

Sprinkler-induced "storms" provided the ability to control the intensity, duration and timing of the inflows at the study sites. Impact-type sprinklers, supplied by the City of Austin Parks and Recreation Department, were used during the tests with the City's fire hydrants used as the source of water. Field observations indicated a spectrum of spray droplet sizes, ranging from fine mist at the periphery of the spray stream to large droplets in the center. A similar range of drop sizes were observed during natural storm events. The number of sprinkler heads were varied for each induced storm and care was taken in placement of the heads to provide uniform coverage of the lot. A schematic of the sprinkler application is presented in Fig. 5. Equivalent rainfall estimates were obtained by placing eight wedge type rain gages on wooden stands around the test lot. During the tests, readings of the rain gages were made at regular intervals (every 15 to 30 minutes) and at the conclusion of the event. Once the individual rainfall totals were compiled, the values were averaged to provide an approximation of the total event volume.

The gravel trench lot was too large for sprinkler coverage, so 2000-gallon (7600-l) capacity rear-end-dispensing water trucks provided by the City of Austin were used. The trucks drove slowly across the upper end of the lot releasing water at approximately 300 gallons per minute (19 l/s). Different event intensities were obtained by varying the number of trucks used, trips made, and number of trucks releasing at one time.

RESULTS OF THE MONITORING SURVEYS

Hydraulic performance results obtained in the stormwater surveys are discussed below. A summary of hydraulic characteristics of the pavements during each runoff event is presented in Table 1.

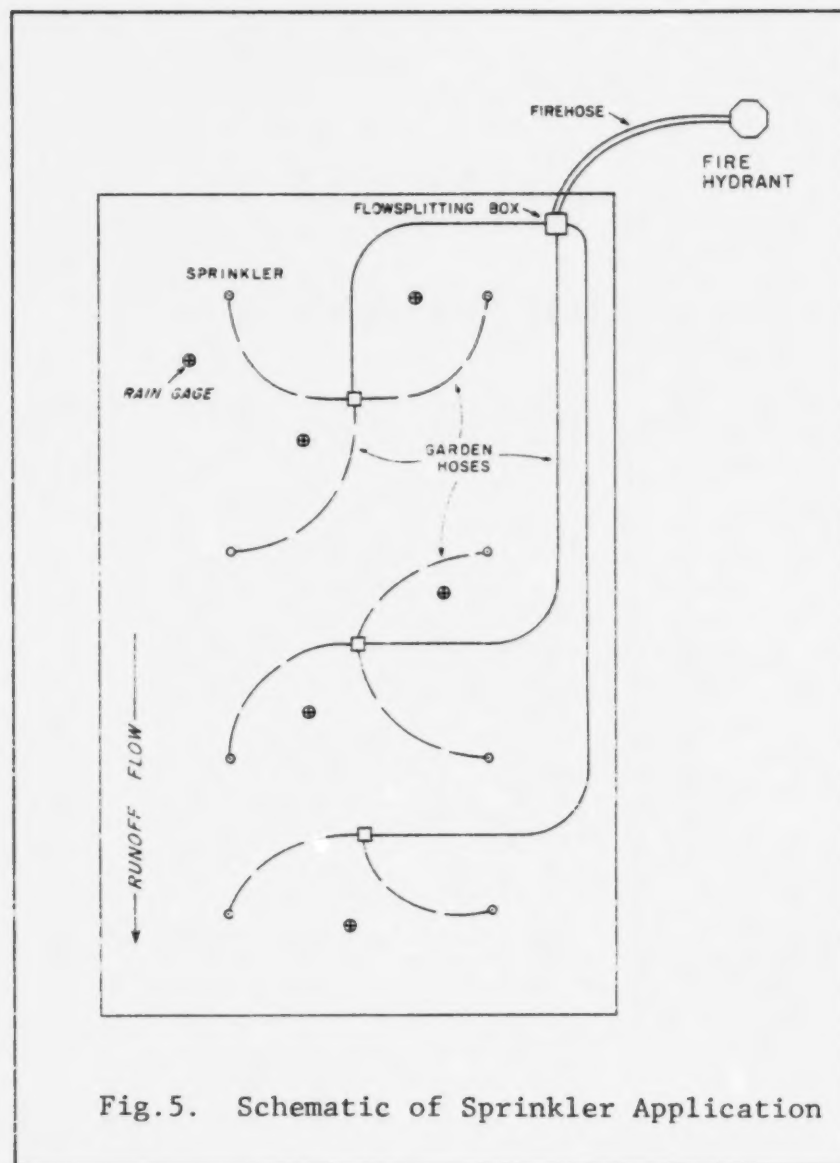


TABLE 1
HYDRAULIC SUMMARY OF STORMWATER SURVEYS

Pavement Type	Event Date	No. of Sprinklers ¹	Total Inflow (in)	Duration (min)	Average Intensity (in/hr)	Peak Discharge (cfs)	Time to Peak (min)	Total Discharge (in)	Runoff Ratio (in/in)	Antecedent Rainfall ² 7-Day (in)	Antecedent Rainfall ² 14-Day (in)
Porous Asphalt ⁴	03/22/82	8	0.94	60	0.94	0.269	58	0.58	0.73	0.02	0.03
	06/01/82	8	1.53	55	1.67	0.237	53	0.56	0.37	0.00	0.00
Gravel Trench	03/03/82	3	0.64	94	0.41	0.440	60	0.49	0.76	4.03	4.03
	03/19/82	4	0.64	70	0.56	0.580	66	0.41	0.64	0.03	0.03
	04/04/82	4	0.64	60	0.65	1.667	55	0.49	0.77	0.12	1.31
Asphalt	06/03/81	*	0.34	46	0.44	0.84	53	0.40	1.18	2.48	7.26
	05/11/82	8	0.21	10	1.26	0.22	7	0.15	0.71	0.99	1.03

1 in = 2.54 cm
1 cfs = 28.31 l/s
1 in/hr = 2.54 cm/hr

¹Values for the gravel trench are the number of water trucks used.
²Runoff-to-rainfall ratio.

³Precipitation amounts recorded at the Austin Airport within the indicated number of preceding days.

⁴Discharge results influenced by infiltration lines along trenches.
* Denotes natural precipitation event. The remainder were sprinkler-induced events.

Porous Asphalt Lot

A maximum intensity of 1.67 in/hr (3.5 cm/hr) was achieved with no resulting surface runoff at the porous asphalt lot. As presented in Table 1, the total discharge volume, the time to peak flow and the peak discharge rates were similar for each event although the inflow varied from 0.94 in (2.4 cm) to 1.53 in (3.8 cm) and the nominal intensity varied from 0.94 in/hr (2.4 cm/hr) to 1.67 in/hr (4.2 cm/hr). Observed base runoff ranged from 37 to 73 percent of recorded sprinkler inflow. Runoff ratios less than unity were attributed to wetting of the base media, storage within the base layer and percolation along the trenches.

Gravel Trench Lot

Table 1 includes a summary of the sprinkler events monitored at the gravel trench lot. The application rates were not varied enough to produce significantly different discharge characteristics. Observed runoff ranged from 64 to 77 percent of recorded inflow, with an average of 72 percent for the three events. Observations made during storm events indicated the small diameter surface gravel was impeding the vertical flux of water, i.e., runoff was flowing across the top of the trench.

Conventional Asphalt Lot

A sprinkler-generated runoff event and a natural precipitation event were monitored at the conventional asphalt study lot, and the results are summarized in Table 1. Estimated runoff volumes ranged from 71 to 118 percent of recorded rainfall. The runoff ratio greater than unity was attributed to rainfall measurement error.

STORMWATER SIMULATION RESULTS

Stormwater hydraulics for each pavement type were simulated with PORPAV. PORPAV was calibrated for each lot using the initial set of observed runoff data. The remaining events were subsequently simulated as verification. A list of PORPAV input data is presented in Table 2. Pavement characteristics such as pavement length, width and depth were obtained from onsite or construction measurements. Other parameters such as the Manning's roughness coefficient, volume of dead storage on the pavement and pavement porosity were estimated. Records of observed inflow were input to PORPAV. Calibration of the model was initialized by varying values of the estimated parameters to reproduce the observed runoff volume. Generally this was accomplished by adjusting the base void volume (the product of depth and porosity) for the pervious lots and the volume of surface storage for the impervious lots. The second objective was to reproduce the observed peak runoff rate. Variations in runoff rates were effected by varying the estimates of average surface slope and the roughness coefficient. For the porous asphalt and gravel trench lot the coefficient of permeability for the base layer was varied to reproduce the observed peak base discharge rate. Results of the simulations are discussed below. A comparison of simulated and observed hydraulic characteristics is summarized in Table 3.

TABLE 2
INPUT DATA FOR PORPAV SIMULATIONS

Element	Parameter	Unit
Rainfall rate	Magnitude	in/hr
Pavement surface	Length	ft
	Width	ft
	Slope	ft/ft
	Area	ft ²
	Permeability	in/hr
	Depth	in
	Porosity	ft ³ /ft ³
	Initial dead storage	in
	Manning's n coefficient of roughness	ft ^{1/6} /s
Pavement base	Permeability	in/hr
	Depth	in
	Porosity	ft ³ /ft ³
	Initial and dead storage	in
	Collection drain capacity	in/hr
Natural soil	Initial and final infiltration rate	in/hr
	Horton's infiltration decay coefficient	hr ⁻¹

TABLE 3
SIMULATION RESULTS
POROUS ASPHALT LOT

	Peak Flow (cfs)	Time to Peak (min)	Runoff Volume (cubic feet)	Detention Time (min)
POROUS ASPHALT LOT - STORM 1				
Observed	0.27	58	745	42
Simulated	0.273	50	745	28
Deviation	+0.003	-8	0	-14
POROUS ASPHALT LOT - STORM 2				
Observed	0.24	53	721	42
Simulated	0.514	55	1,409	25
Deviation	+0.274	+2	+688	-17
GRAVEL TRENCH LOT - STORM 1				
Observed	0.44	60	1,960	29
Simulated	0.497	90	2,107	24
Deviation	+0.057	+30	+147	-5
GRAVEL TRENCH LOT - STORM 2				
Observed	0.58	66	1,693	24
Simulated	0.487	70	1,650	24
Deviation	-0.093	+4	-43	0
GRAVEL TRENCH LOT - STORM 3				
Observed	1.67	55	1861	19
Simulated	0.472	60	1,448	23
Deviation	-1.198	+5	-413	+4
CONVENTIONAL ASPHALT LOT - STORM 1				
Observed	0.34	53	308	1
Simulated	0.297	50	269	5
Deviation	-0.043	-3	-99	+4
CONVENTIONAL ASPHALT LOT - STORM 2				
Observed	0.22	7	138	5
Simulated	0.257	10	140	8
Deviation	+0.037	+3	+2	+3

1 cfs = 28.32 lps

1 ft³ = 28.32 l

Porous Asphalt Lot

The discharge hydrograph for the calibrated data set, presented in Fig. 6a, accurately resembles the observed one, but is advanced about fifteen minutes, as is reflected in the difference in detention times. The calibrated coefficients of dead storage and base permeability were held constant during the simulation of the remaining event. Observed and simulated discharge hydrographs for the final event are displayed in Fig. 6b. The significant overestimation of peak discharge rate and volume probably resulted from an incorrect sprinkler inflow measurement (used as input to PORPAV) or an inconsistent hydraulic response, possibly increased base storage or percolation.

Gravel Trench Lot

Three artificial rainfall events were simulated for the gravel trench lot. For each case the runoff from the conventional asphalt lot was simulated with PORPAV and subsequently used as input to the gravel trench simulation, hence, characteristics for both facilities had to be determined. A summary of simulation results is presented in Table 3. Coefficients of base permeability and dead storage on the asphalt lot and in the gravel trench were determined by calibrating PORPAV with the initial runoff event data. The calibration hydrograph is presented in Fig. 7a. Runoff characteristics were reproduced quite well for the second event, as displayed in Fig. 7b. Incorporating the same physical characteristics of the asphalt and gravel trench for the final event simulation yielded a less satisfactory comparison, presented in Fig. 7c. The major discrepancies in the two hydrographs of the third event occur during the periods of water release from the water trucks, possibly an artifact of utilizing a constant inflow rate during this period in the simulation.

Asphalt Lot

Simulation results for the conventional asphalt lot are compared with recorded values in Table 3. The second event was used to calibrate PORPAV and was accurately simulated, as shown in Fig. 8a. The simulated hydrograph for the second event is compared to observed results in Fig. 8b. The simulation did not reproduce the peak discharge rate, possibly a result of a short, intense burst of rain which was undetected in the rainfall data. Both simulations depicted the very rapid detention times associated with the asphalt lot events.

CONCLUSIONS

The revised PORPAV satisfactorily simulated sprinkler-generated storm-water hydraulics of both porous and nonporous pavement facilities. Sprinkler application rates ranged from 0.4 to 1.7 inches per hour. The favorable simulation results obtained in this study suggest that PORPAV can be used to assess the relative hydraulic performance of pavement facilities available for urban runoff control. The option of using a drain pipe for the base layer discharge has not been evaluated. A future study should be conducted at an existing pavement facility utilizing a collection drain system to assess the ability of PORPAV to simulate such a control strategy.

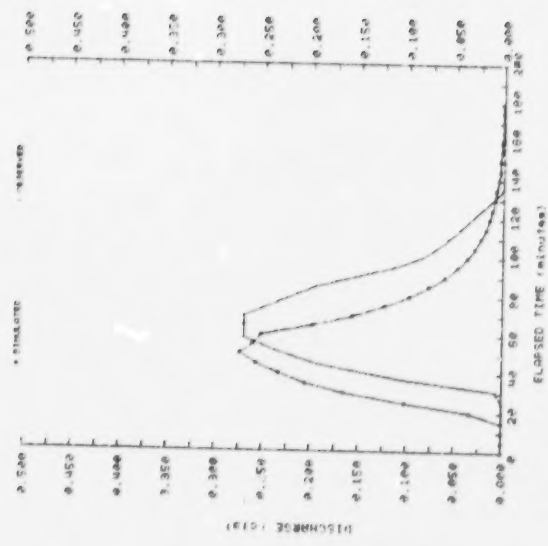


FIG. 6a SIMULATED VS. OBSERVED HYDROGRAPHIC FOR 3-24-62.

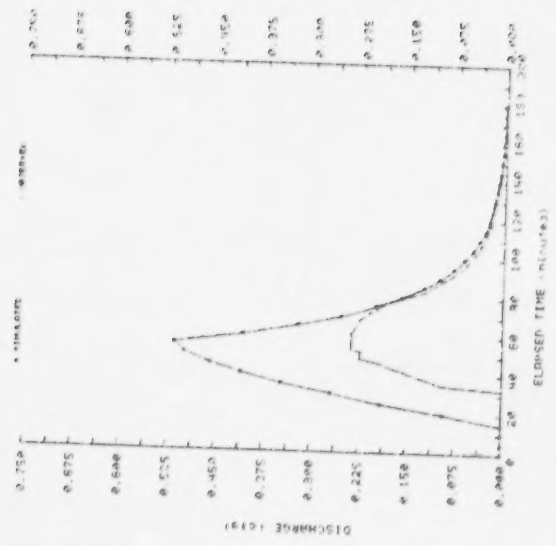


FIG. 6b SIMULATED VS. OBSERVED HYDROGRAPHIC FOR 4-1-62.

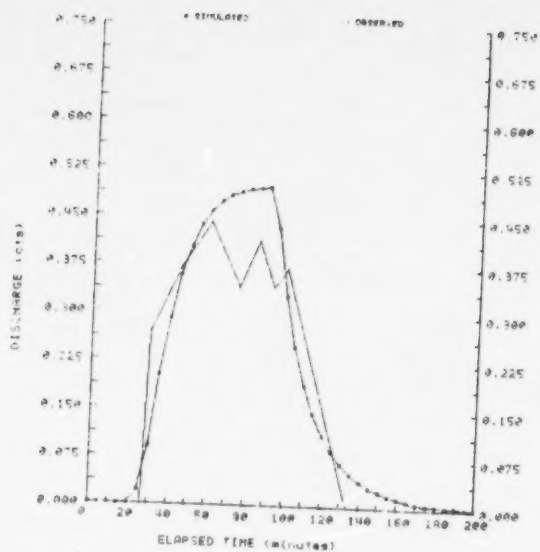


FIG. 7a SIMULATED VS. OBSERVED HYDROGRAPHS FOR 3/3/82.

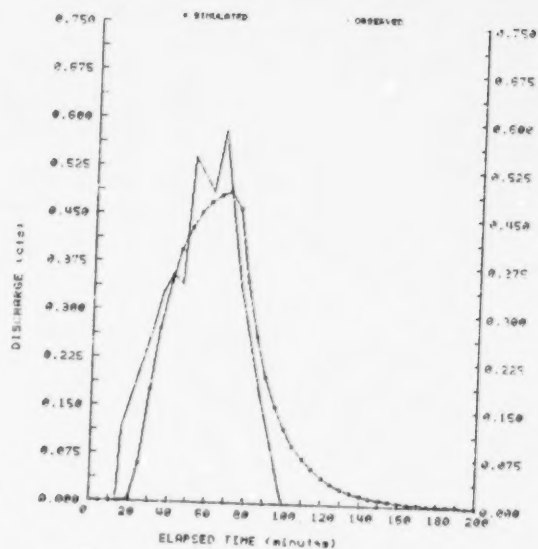


FIG. 7b SIMULATED VS. OBSERVED HYDROGRAPHS FOR 9/18/82.

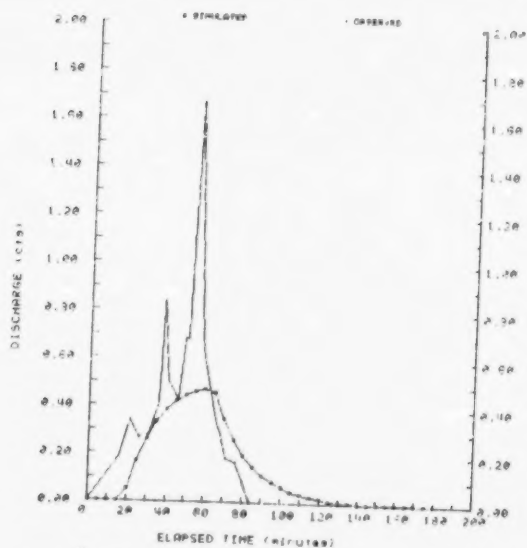


FIG. 7c SIMULATED VS. OBSERVED HYDROGRAPHS FOR 4/4/82.

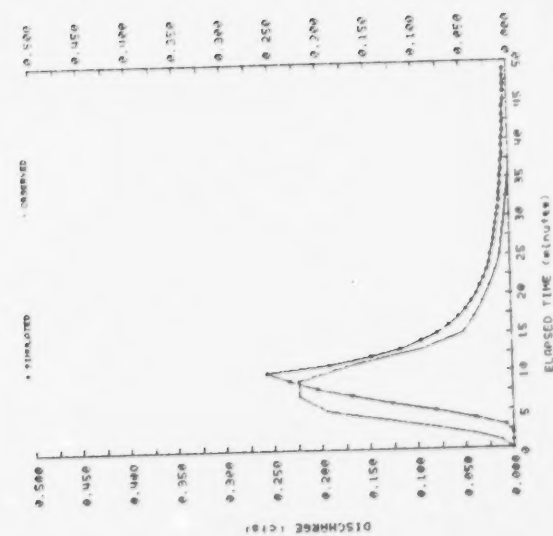


FIG. 8a SIMULATED VS. OBSERVED HYDROGRAPHS FOR 5/11/92.

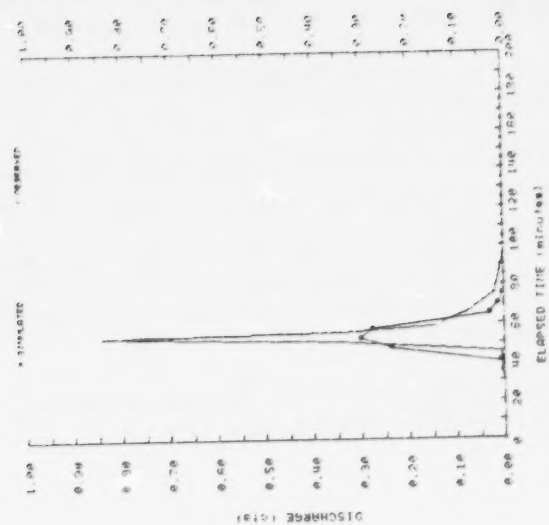


FIG. 8b SIMULATED VS. OBSERVED HYDROGRAPHS FOR 5/11/92.

REFERENCES

1. Biggers, D. J., J. P. Hartigan, Jr. and H. A. Bonuccelli, Urban Best Management Practices (BMP's): Transition from Single-Purpose to Multipurpose Stormwater Management. Conference Paper, International Symposium on Urban Storm Runoff, Univ. of Kentucky, Lexington, Kentucky - July 28-31, 1980.
2. Diniz, E. V. Porous Pavement Phase I - Design and Operational Criteria, EPA-600/2-80-135, May 1980.
3. _____. Storm Water Management Model Supplement to the User's Manual, EPA Project No. CR-805664, March 1981.
4. Heaney, J. P., W. C. Huber, M. A. Medina, Jr., M. P. Murphy, S. J. Nix and S. M. Hasan. Nationwide Evaluation of Combined Sewer Overflows and Urban Stormwater Discharges, Volume II: Cost Assessment and Impacts, EPA-600/2-77-064, March 1977.
5. Israelson, O. W. and V. E. Hansen, Irrigation Principles and Practices, 3rd Edition, John Wiley and Sons, Inc., New York, N.Y., 1962.

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

PLANNING AND IMPLEMENTATION OF REGIONAL STORMWATER
MANAGEMENT FACILITIES IN MONTGOMERY COUNTY, MARYLAND

JANUARY 1983

John M. Crouse, P.E.,¹ Vincent H. Berg, P.E.,² and Linda J.D. Mitchell³

I. INTRODUCTION

Montgomery County, Maryland, is located to the north and the west of Washington, D.C. The County is geologically located just above the fall line between the Piedmont Plateau and the Coastal Plain. The County is characterized by gently rolling topography with steep slopes adjacent to the major stream valleys. Most of the major streams originate in the County and drain into the Potomac River. The watersheds generally range in size from 20 square miles to approximately 130 square miles. The County contains approximately 500 square miles.

Being adjacent to the Nation's Capital, the County has experienced a major growth in population and development over the last 30 years. The population in 1950 was approximately 160,000 and today it is approximately 600,000. The majority of the population is concentrated along the major transportation corridors extending out like spokes of a wheel from Washington, D.C. These corridors do not follow watershed boundaries, but extend across several watersheds as they radiate out from the District of Columbia.

Each year, approximately 2,000 to 2,500 acres are converted from rural to urban land use to house the population increase and satisfy demographic changes. As a result of this urban growth, it is estimated that approximately 200 miles of the County's 1,015 miles of streams have suffered deterioration in the form of increased flooding, accelerated channel erosion, and reduced water quality. An additional eight miles are estimated to be affected each year.

- ¹ Project Manager, Water Resources Section, Greenhorne & O'Mara, Incorporated, Riverdale, Maryland.
- ² Senior Environmental Engineer, Water Resources Section, Department of Environmental Protection, Montgomery County, Maryland.
- ³ Former Project Engineer, Water Resources Section, Greenhorne & O'Mara, Incorporated, Riverdale, Maryland.

II. STORMWATER MANAGEMENT (SWM) PROGRAM DEVELOPMENT

Stormwater management was first required as an outgrowth of sediment control requirements. In July 1961, the State Attorney General interpreted the powers and duties of the State Water Pollution Control Commission to include regulation of sedimentation resulting from land development activities as an industrial pollutant.

Throughout the late 1960's and early 1970's local jurisdictions and the State General Assembly adopted legislation requiring sediment control during land development operations. In 1971, the Maryland Attorney General made another landmark interpretation which said that the state and local jurisdictions, under sediment control powers, could require the control of runoff after development in a manner to prevent off-site erosion.

In July 1971, the Montgomery Soil Conservation District (MSCD) adopted its first stormwater management policy. This policy required the detention of stormwater runoff from the two-year storm event on each development site. The release rate could be no greater than the runoff that would have left the site prior to development. To achieve these criteria, the majority of the developers used on-site stormwater detention ponds as the simplest and least expensive means of compliance. However, where the site, due to size, topography, or the value of the land, did not lend itself to a surface impoundment, other means have been used, such as rooftop detention, underground vaults and pipes, infiltration pits, surface storage on parking lots and pervious parking lots.

The action of the MSCD in requiring on-site detention facilities in conjunction with new development was a significant step forward in achieving better control of storm runoff. Requiring developers to plan for stormwater detention recognizes the public responsibility to regulate the allowable peak runoff associated with land development. However, reliance on only this approach to achieve the stormwater management objectives is viewed as only a partial solution for the following reasons:

1. In many instances, individual development proposals have site conditions which preclude the installation of truly effective on-site controls. However, until recently, no mechanism existed to permit a developer to contribute, as an alternative solution, to a more strategically located facility that could provide a much higher level of stormwater control and which is more cost effective.
2. In some instances, greater stormwater control is desirable at particular locations then can be required under existing MSCD criteria.
3. While certain types of control structures are effective in meeting present MSCD requirements for on-site controls, they may be highly undesirable from other standpoints such as public safety or aesthetics.

4. By themselves, present on-site control requirements are incapable of handling runoff volume and velocities accompanying storms of greater intensity than the two-year frequency storm.
5. No reliable mechanism for insuring perpetual maintenance of private, on-site facilities exists. Therefore, many facilities fail to function as designed due to a lack of maintenance.

In addition to the above considerations, a review of stormwater management plans for on-site facilities, reviewed and approved by the MSCD during the 12-month period from October 1, 1978, through September 30, 1979, indicated that 78 developments on 1,584 acres would expend over \$3.5 million to provide on-site stormwater management.

III. CURRENT PROGRAM DIRECTION

In recognition of the growing problems created by uncontrolled urban storm runoff and the limitations of the ongoing program of on-site detention of the two-year frequency storm, the County embarked upon an aggressive program to provide stormwater management protection on a watershed basis. In May 1973, the stormwater management program was established within the Montgomery County Department of Environmental Protection (DEP) to develop and implement a County-wide stormwater management program. The program consists of two major thrusts, first, the implementation of remedial projects, and, second, the development of preventive stormwater management projects on a watershed basis.

A. Remedial Projects

The remedial focus of the County SWM program includes projects to correct existing problems caused by uncontrolled runoff from previously developed areas. These projects are designed to help stabilize conditions in critical deteriorating stream reaches.

B. Preventive Programs - On a Watershed Basis

While the remedial efforts serve to control or eliminate existing problems, they do little to fulfill the need for comprehensive, long-range stormwater management. Experience has taught us that controls for storm runoff never realize the optimum potential until we consider the watershed as a whole and evaluate the effects that proposed land use changes will have on the quantity of storm flows throughout the watershed. In an effort to fulfill this need, Montgomery County has embarked upon an ambitious program of planning stormwater management on a watershed basis.

In 1973, the County Council authorized funds for consultant studies of three major watersheds within the County. Several common conclusions emerged from the three studies:

1. Control of runoff should be accomplished near the source either on each development site, or near the site where several individual developments could share one facility.

2. The level of control should be expanded to include not only the 2-year storm, but also the 10-year storm, and, where appropriate, up to the 100-year storm.
3. Needs for non-point source pollution control should be evaluated and appropriate controls implemented.

These studies also indicated that stormwater management controls (primarily detention impoundments) were most effective when located in or near the headwaters. Conversely, detention structures placed on tributary sub-watersheds near the mouth of the watershed had little benefit and could actually reinforce peak flows due to timing of the release of the detention structure to coincide with the peak flow of the main stream. For this reason it has become evident that the watershed must be considered as a whole when planning an effective program.

As a result, the on-site stormwater management program has undergone several changes. The major change is to allow a developer the option of contributing towards the costs of a central (public) facility when an on-site facility is deemed less appropriate than a central facility. This approach allows the developer more flexibility in designing his proposed development and enhances the County's ability to pool resources from several developments for a more cost-effective solution, while providing the County with the flexibility in selecting the best sites for central stormwater management facilities.

IV. STORMWATER MANAGEMENT ORDINANCE

The culmination of these long years spent in developing the County's SWM program came on March 3, 1980, with the signing of the new County Code amendment, which became effective on June 2, 1980. The Ordinance assigned to Department of Environmental Protection (DEP) the responsibility of coordinating SWM in the County.

A. Legislative Intent

1. All developers are intended to be held responsible for the impacts of storm runoff created by their development. This has been accomplished by requiring the installation of on-site detention on all new development or, in the event the on-site requirement is waived, through the payment of a "contribution" or fee not greater than the cost of on-site SWM.
2. The contribution structure is such that lighter-density development is rewarded. The Ordinance provides for the collection of SWM fees based on typical impervious areas and storage quantities required for excess runoff rates, which are proportionately lower for less dense development. In fact, construction of single-family residences on lots of two acres or more are completely exempt from the provisions of the Ordinance.

3. The planning and building of small, on-site structures is discouraged. Many of these on-site detention structures have proven to be relatively expensive to build, and most take up usable land area. Most of these structures have been transferred to Home-owners Associations basically uninterested and incapable of proper maintenance, and consequently they have not received the required maintenance.
4. The Ordinance provides for DEP to develop operational plans for centralized SWM structures to be placed in strategic locations. It also provides for County participation to assist developers of strategic sites to design and build regional structures with greater than required storm runoff controls.
5. To assure fulfillment of SWM controls within a watershed with the onset of development in a given watershed, a program for construction of public SWM facilities has also been implemented through the Capital Improvement Program (CIP).
6. The Ordinance provides for the coordination of proposals for potential and needed central SWM sites to be incorporated into updated versions of master planning documents.

B. Regional Stormwater Management

The new legislation, while encouraging SWM on all developments, relies heavily on the use of centralized SWM facilities, each of which serves a number of developments and possibly provides multi-use opportunities for the community. The SWM legislation has had a marked effect on reducing the number of small on-site SWM structures. Under the County's stormwater management ordinance small on-site structures can be waived, and larger centralized SWM structures can be built, either through the County's CIP Program or through developer-constructed SWM projects with County participation. This combination system of regional and on-site SWM structures is most effective when SWM facilities are strategically placed in watersheds which will have the greatest benefit for the environment as well as the community.

When development pressure is increasing within a major subwatershed the Department of Environmental Protection can conduct a detailed study of a subwatershed utilizing CIP funds under the Preliminary Stormwater Management Investigation Project. The major purpose of these studies is to determine the strategic location of regional stormwater management facilities and to develop a detailed operational plan for the subwatershed. The subwatershed studies will generally be for areas of two to four square miles.

To date, five of these studies have been completed and two additional studies are now being conducted. The Cabin Branch study is typical of these preliminary stormwater management studies.

V. CASE STUDY - CABIN BRANCH

A. Introduction

Cabin Branch is a tributary of Great Seneca Creek in the Potomac River basin. Cabin Branch is located on the north side of Gaithersburg, the second largest city in Montgomery County. The location of the study area is shown on Figure 1. The drainage area of the studied portion of Cabin Branch is approximately 4-1/4 square miles.

The Gaithersburg area has been and is expected to be an area of high growth in the County. The effects of such growth have been partially controlled by sediment control and required on-site stormwater management facilities.

The Montgomery County Department of Environmental Protection contracted with the consulting firm of Greenhorne & O'Mara, Incorporated, to perform a preliminary investigation of stormwater management, flood control, and erosion control requirements for the existing and projected conditions of the Upper Cabin Branch watershed. The purpose of the study was to determine the larger-scale effects of on-site SWM facilities on the receiving stream, to evaluate and locate regional SWM facilities, and to evaluate the impact of multiple-frequency control on stream discharges.



Figure 1. Vicinity Map

Excerpts of the report on the investigation presented in this paper illustrate the process of analyzing watersheds and locating regional stormwater management facilities.

B. Data Acquisition

Extensive data from such County agencies as the Office of Planning and Capital Programming and Department of Transportation along with information from the Montgomery Soil Conservation District, the Maryland-National Capital Park and Planning Commission, the Washington Suburban Sanitary Commission, and the Federal Emergency Management Agency was assembled. This information included proposed capital improvements, bridge and culvert crossings, hydrologic soil groups, existing stormwater management facilities, master plans, flood plain delineation, and utility locations. Other data such as aerial photography taken at various periods, tax maps, topographic maps, and historical sites atlases proved valuable in developing predeveloped land use (used as baseline data), existing land use, and projected land use mapping. The information obtained from these data sources was checked with the field conditions, and modifications or additions to the data were made where necessary.

C. Methodology

1. Hydrology model

The hydrologic analysis of the watershed was accomplished by utilizing the Soil Conservation Service Technical Release Number 20 Computer Model. The hydrologic condition of the soils was taken from the SCS Soil Survey for Montgomery County. Cover conditions were determined for predeveloped (circa 1942), existing, and projected land use conditions through the data acquisition process previously described.

The Cabin Branch study area was approximately one percent developed in the early 1940's, consisting mostly of farmhouses and farmsteads. Approximately seventy percent of the area was in cultivation, approximately twenty percent was forested, and the rest of the watershed was pasture or open space.

Currently, the watershed is undergoing rapid residential development, generally of high density. Approximately fifty percent of the study area is now developed. About twenty-five percent is still in cultivation. Much of the stream valley of Cabin Branch contains open space.

Approximately eighty percent of the study area is projected to be ultimately developed. The remainder is zoned to be park land, golf course, and other open space. The area in the vicinity of the airport is planned to be developed as commercial/light industrial sites. The extreme northeast corner of the study area will be developed as single-family residential.

2. Subarea delineation criteria

In order to provide stream flow estimates at several points along Cabin Branch, the watershed was divided into subareas according to runoff flow paths and velocities and points of discharge into the stream. Drainage area divides were first drawn between direct tributaries to Cabin Branch which were shown on the USGS 7 1/2-minute quadrangles as perennial streams. If forks or tributaries of the direct tributaries were also indicated as perennial streams, the subdivides between them were then delineated. If very dissimilar land uses were encompassed in a subarea, then the area was further subdivided. Discharge points used in other studies and such landmarks as major road crossings were also used to separate the different subareas, as well as major existing SWM facilities or potential sites for facilities. No subarea was to be greater than fifteen percent of the study area.

The final analysis for Cabin Branch was performed using eighteen basic drainage subareas as shown on Figure 2. These subareas were further subdivided as necessary to account for additional road crossings and significant SWM facilities in existence. A schematic diagram of the study area hydrology is shown in Figure 3.

3. Key point selection

Seven sites of interest called key points were located along the main stem of the study stream to serve as convenient points at which to compare the predeveloped, existing, and projected discharges. Easily recoverable locations such as road crossings and confluences of the main stem and major tributaries were designated as key points as well as other strategic locations such as areas where land use has undergone or will undergo significant change.

4. Existing SWM facility screening criteria

Data on the existing SWM facilities were compiled from the previously mentioned data files and field reconnaissance. An extensive amount of information was available because the watershed has undergone urbanization since the inception of the sediment control and SWM regulations and the requirement for submission of plans and computations. Screening criteria were developed to remove from the investigation certain SWM facilities that were believed to have little impact for the scale of the investigation.

Dry and wet ponds were considered in the hydrologic analysis. Other SWM measures (such as rooftop detention, gravel trenches, and seepage pits) found in the watershed were not of sufficient size to achieve significant stormwater management.

It is interesting to note that the two SWM ponds in subarea 10 are in a subdivision currently under construction and the ponds are being built under the County's participation program. Wet and dry ponds were considered not significant and were excluded from the hydrologic analysis of the

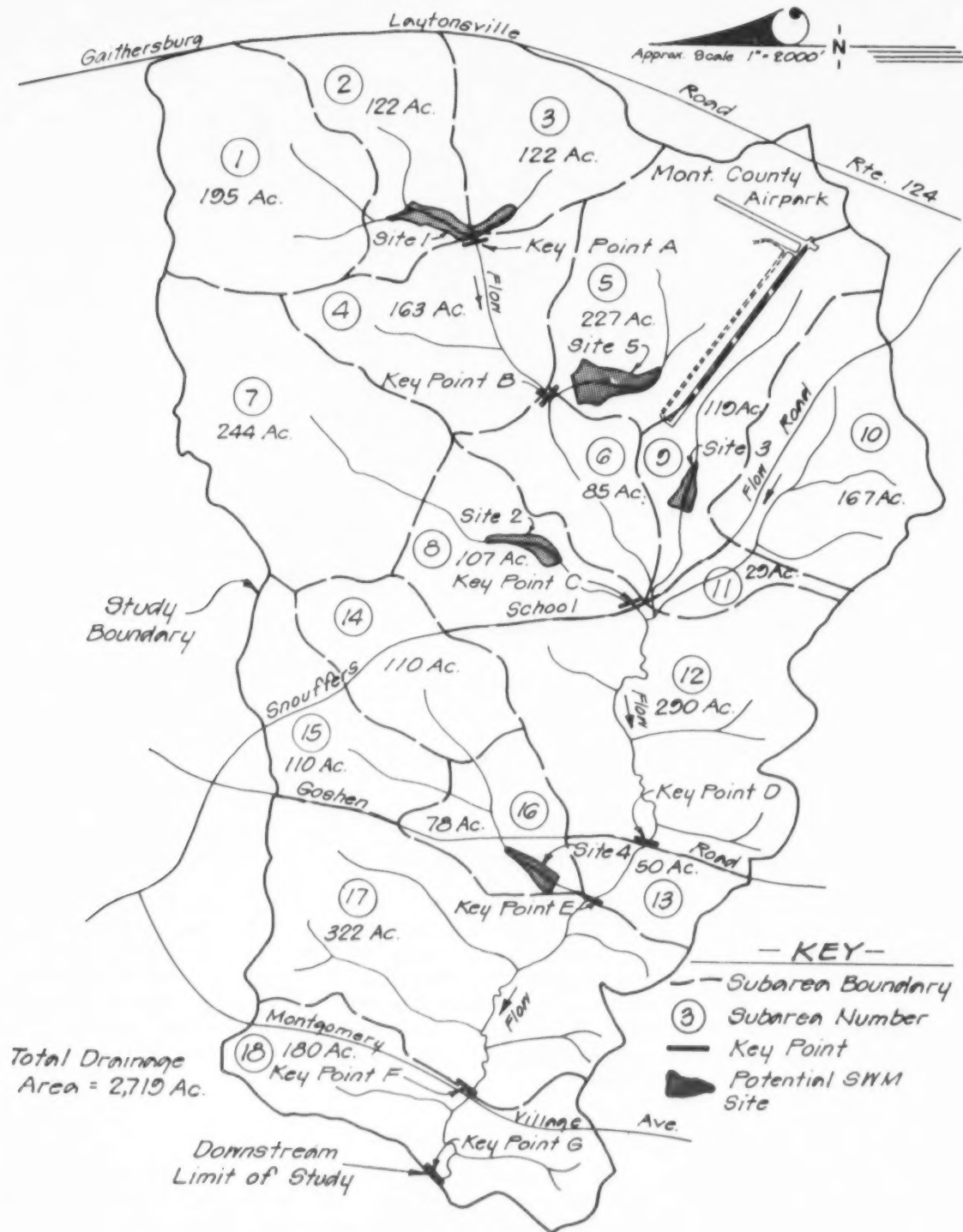


Figure 2. Drainage Areas and Subareas

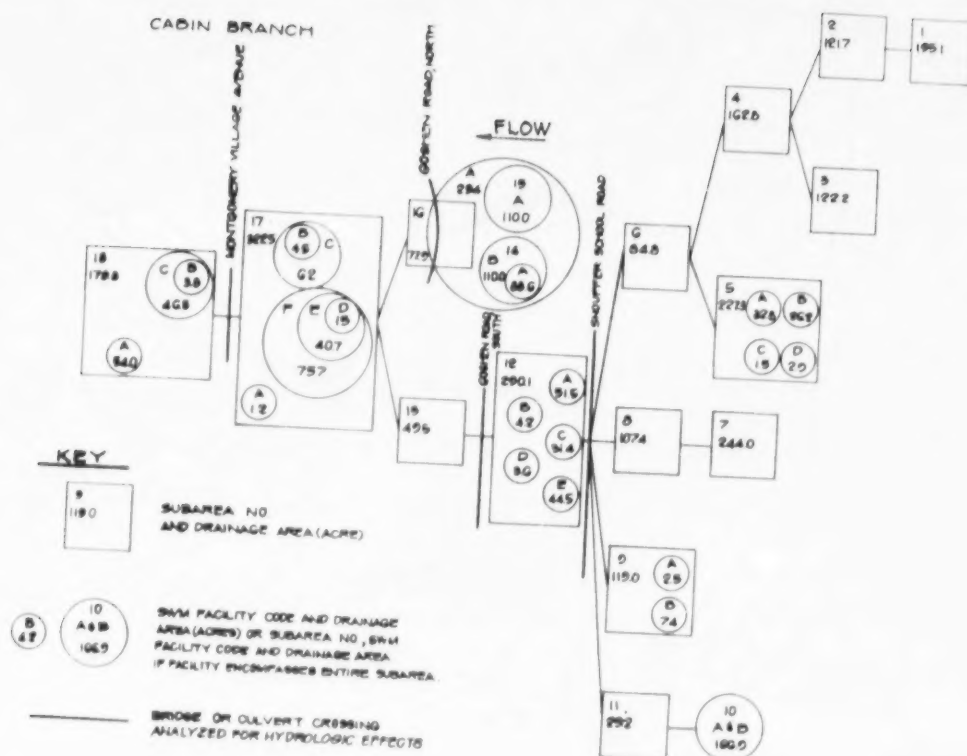


Figure 3. Schematic Diagram of Hydrology

watershed if the pond drainage area was less than five percent of the sub-area in which the facility was located and less than five-tenths percent of the entire study area. If the drainage areas for the ponds which were screened out by these criteria summed to more than five percent of the sub-area in which the ponds were located or to more than one percent of the total study area, the largest excluded structure was reincluded. An excluded facility may be assumed to have little impact on the watershed as a whole but does provide some protection against erosion and flooding in the immediate downstream vicinity. Eighteen ponds in the Cabin Branch study area were determined to be significant; eight smaller ponds were dropped from consideration.

5. Peak discharge determination

Estimates of the peak discharges in the watershed for the 2-, 10-, and 100-year storms for predeveloped, existing, and projected conditions were produced using the SCS TR-20 computer model with the input data previously described and rainfall from the U.S. Weather Bureau Technical Paper Number 40. The peak discharges are summarized in Table 1 for key points along the stream.

Table 1. Peak Discharges at Key Points

Key Point*	2-Year			10-Year			100-Year		
	Predev.	Ex.	Proj.	Predev.	Ex.	Proj.	Predev.	Ex.	Proj.
A	154	219	326	495	609	763	955	1110	1297
B	210	304	364	654	811	905	1246	1455	1586
C	331	626	837	1030	1542	1906	1976	2683	3205
D	420	720	923	1252	1666	2184	2479	3221	3910
E	440	718	866	1209	1786	2246	2619	3616	4320
F	482	771	920	1281	1916	2373	2774	3819	4506
G	500	793	941	1313	1918	2343	2837	3661	4230

*Key Point locations are as follows:

- A - downstream of subareas, 1,2, and 3, in headwaters
- B - downstream of subareas, 1,2,3, and 4, north of Montgomery County Airpark, near Green Farm
- C - downstream of subareas, 1-9, at Snouffers School Road
- D - downstream of subareas, 1-12, at Goshen Road
- E - downstream of subareas, 1-16
- F - downstream limit of subarea 17, at Montgomery Village Avenue
- G - downstream limit of study area

6. Runoff increase criteria

To determine sites where SWM basins may be most beneficial, the percent change in discharge from existing to projected conditions was examined for each key point. Subareas with relatively high increases in discharge were considered for SWM facility sites.

7. Site considerations

The percent increase analysis indicated that key points A and C should be explored for SWM sites. A significant increase in peak discharge from existing to projected conditions was also shown at key point D, but no feasible basin sites were apparent in that area. Five sites were analyzed: site 1 was located at key point A; sites 2, 3, and 5 were situated upstream of key point C on tributaries to Cabin Branch; and site 4 was at a location for which a preliminary design had been completed for another study.

Potential sites were evaluated in terms of the downstream land uses, the timing of the peak discharge relative to the times of downstream peaks, and the natural storage available at the site without unduly infringing on the surrounding land uses. Soils in the immediate vicinity of the sites were examined for suitability as embankment material and for reservoir lining.

Factors other than land use, soil type, hydrologic response, and hydrologic performance serve an important role in SWM facility location study. Acquisition of property, construction cost, and utility conflicts are three such factors with potentially high impacts. The impacts of these factors for the five proposed sites were evaluated and are indicated in Table 2.

Table 2. SWM Site Considerations

Factor	Specifics	Comments
<u>Site 1</u>		
Area Property Owners	MNCP&PC Hadley Farms Dairy, Inc. and First Citizens Development Corp. et al	Favorable
Construction Cost	\$111,300	Moderate
Project Area	11.18 acres (MNCP&PC); 2.68 acres (Hadley); 1.47 acres (First Citiz.)	
Access Right-of-Way	0.35 acres (MNCP&PC); 1.29 acres (First Citiz.)	
Utilities	8-inch, 10-inch and 12-inch sanitary sewers (1450 L.F.)	Requires relocation or protective measures
<u>Site 2</u>		
Area Property Owners	H.J. Bobys, et al	Favorable if developed by owner
Construction Cost	\$92,000	Moderate
Project Area	11.03 acres	
Access Right-of-Way	0.35 acre	
Utilities	Apparently none	Very favorable
<u>Site 3</u>		
Area Property Owners	Montgomery County Airpark, Inc. and A.W. & E. Stang	Favorable if developed by owner
Construction Cost	\$83,000	Moderate
Project Area	11.49 acres (MCA); 0.80 acres Stang	
Access Right-of-Way	0.41 acres (Stang)	
Utilities	Apparently none	Very favorable
<u>Site 4</u>		
Area Property Owners	C.J. and I.B. Savage	Favorable if developed by owner
Construction Cost	unknown	Pond privately proposed
Project Area	unknown	
Access Right-of-Way	unknown	
Utilities	10-inch sanitary sewer	Requires relocation or protective measures
<u>Site 5</u>		
Area Property Owners	MNCP&PC, J.E. Richardson Trus., and Montgomery Co. Revenue Auth.	Favorable if developed by owner
Construction Cost	\$93,800	Moderate
Project Area	8.46 acres (MNCP&PC); 3.22 acres (Richardson); 1.30 acres (MCRA)	
Access Right-of-Way	0.06 acre (J.E. Richardson Trus.)	
Utilities	8-inch sanitary sewer (1920 L.F.)	Requires relocation or protective measures

8. Proposed pond development

Preliminary designs for sites 1 and 2 were based on providing reduction of the 2-, 10-, and 100-year storms from the projected peak levels to approximately the lesser of predeveloped or existing levels. The basin at site 3 was sized based on the intents of the potential developer of the area as discussed with the engineer retained by the developer. Site 4 was based on file data as obtained from the Montgomery Soil Conservation District. Site 5 design was based on providing 2- and 100-year control to the predeveloped conditions and control to near-predeveloped conditions for the 10-year storm.

Storage requirements at a site were estimated by the methodology presented in the SCS Technical Release number 55, "Urban Hydrology for Small Watersheds." Initial peak discharge reduction was typically accomplished with a low flow orifice/riser/barrel arrangement, with computations based on the Maryland SCS publication, "Standards and Specifications for Soil Erosion and Sediment Control in Developing Areas." The size and location of the outlet works were adjusted as necessary to attain the desired goals based on the initial design as suggested by the above references.

9. Scenario development and testing

Upon obtaining satisfactory peak discharge reductions for the individual sites by utilizing the TR 20 program, the watershed hydrology model for the projected land use conditions was modified to reflect the individual and combinations of individual proposed sites. Ten scenarios were developed to determine the effects of the five proposed SWM sites. The scenarios and the sites comprising the scenarios were:

- Scenario 1 - SWM Site 1 Only
- Scenario 2 - SWM Site 2 Only
- Scenario 3 - SWM Site 3 Only
- Scenario 4 - SWM Site 4 Only
- Scenario 5 - SWM Site 5 Only
- Scenario 6 - SWM Sites 1 and 2
- Scenario 7 - SWM Sites 1 and 3
- Scenario 8 - SWM Sites 2 and 3
- Scenario 9 - SWM Sites 1, 2, and 3
- Scenario 10 - SWM Sites 1, 3, and 5

The discharge values at the key points for the projected land uses are shown in Table 3 for scenarios 1 to 5. Table 4 presents the percent change in estimated peak discharges from predeveloped, existing, and projected conditions to the proposed conditions at key points for selected scenarios. The more desirable scenarios produce a smaller percent change in discharges for a greater number of key points.

Table 3. Key Point Discharges for Scenarios 1 to 5

Key Point	Existing Structures Only			Proposed Land Use Conditions, with SWM Scenario Number:				
	Predev.	Ex.	Proj.*	1 Basin 1 only	2 Basin 2 only	3 Basin 3 only	4 Basin 4 only	5 Basin 5 only
Two-year storm								
A	154	219	326	155**	321	321	321	321
B	210	304	364	185	357	357	357	357
C	331	626	837	701	778	716	823	596
D	420	720	923	786	860	807	905	704
E	440	718	866	763	818	782	807	714
F	482	771	920	821	870	832	857	766
G	500	793	941	844	890	853	878	789
Ten-year storm								
A	495	609	763	507**	755	755	755	755
B	654	811	905	599	889	889	889	889
C	1030	1542	1906	1501	1696	1666	1875	1451
D	1252	1666	2184	1682	1908	1907	2130	1670
E	1209	1786	2246	1822	1989	1991	2060	1790
F	1281	1916	2373	1945	2093	2086	2167	1897
G	1313	1918	2343	1954	2078	2074	2148	1900
One hundred-year storm								
A	955	1110	1297	963**	1287	1287	1287	1287
B	1246	1455	1586	1180	1560	1560	1560	1650
C	1976	2683	3205	2496	2907	2935	3151	2593
D	2479	3221	3910	3213	3544	3573	3814	3245
E	2619	3616	4320	3582	3916	3953	4174	3565
F	2774	3819	4506	3747	4004	4042	4301	3703
G	2837	3661	4230	3626	3800	3840	4046	3557

*At key points above the proposed SWM sites, the listed peak discharges with and without the proposed SWM may differ slightly due to the use of a larger time increment in modeling the proposed SWM Scenarios.

**Outflow from proposed SWM Basin 1.

Table 4. Percent Change in Key Point Discharges for Selected Scenarios

Scenario 1 (Proposed SWM Facility 1 only)											
KEY POINT	2-YEAR FREQUENCY			10-YEAR FREQUENCY			100-YEAR FREQUENCY				
	Pre-Prop	Ex-Prop	Proj-Prop	Pre-Prop	Ex-Prop	Proj-Prop	Pre-Prop	Ex-Prop	Proj-Prop	Pre-Prop	Proj-Prop
A	1	-29	-52	2	-17	-34	1	-13	-26		
B	-12	-39	-49	-8	-26	-34	-5	-19	-26		
C	112	12	-16	46	-3	-21	26	-7	-22		
D	87	9	-15	34	1	-23	30	0	-18		
E	73	6	-12	51	2	-19	37	-1	-17		
F	70	6	-11	52	2	-18	35	-2	-17		
G	69	6	-10	49	2	-17	28	-1	-14		
Scenario 7 (Proposed SWM Facilities 1 and 3)											
A	1	-29	-52	2	-17	-34	-1	-13	-26		
B	-12	-39	-49	-8	-26	-34	-5	-19	-26		
C	75	-7	-31	21	-19	-34	18	-13	-27		
D	62	-6	-26	22	-8	-30	21	-7	-23		
E	57	-4	-20	-37	-7	-26	25	-9	-24		
F	55	-3	-19	38	-8	-25	25	-9	-23		
G	55	-3	-18	36	-7	-24	20	-7	-20		
Scenario 10 (Proposed SWM Facilities 1, 3 and 5)											
A	1	-29	-52	2	-17	-34	1	-13	-26		
B	-12	-39	-49	-8	-26	-34	-5	-19	-26		
C	8	-43	-57	6	-29	-43	6	-22	-35		
D	13	-34	-49	7	-20	-39	7	-18	-32		
E	24	-24	-37	15	-22	-38	8	-22	-34		
F	24	-22	-35	15	-23	-38	8	-22	-34		
G	25	-21	-34	14	-22	-36	5	-19	-30		

(Pre - predeveloped conditions; Ex - existing conditions; Proj - projected land use conditions; Prop - Projected land use with proposed scenario in place).

D. Conclusions and Recommendation

1. Conclusions

- a. From predeveloped to existing conditions, discharges at key points in the Cabin Branch watershed have increased as much as 89 percent for the 2-year frequency, 50 percent for the 10-year frequency, and 38 percent for the 100-year frequency.
- b. From existing to projected conditions, discharges have been estimated to increase as much as 34 percent, 31 percent, and 21 percent for the same frequencies.
- c. The largest percent increases generally occur in the upper portion of the watershed.
- d. The Snouffers School Road and Goshen Road crossings of the main stem have a significant impact on the peak of the more frequent discharges.
- e. Proposed stormwater management facilities located in the upper portion of the watershed generally reduce the peak discharges throughout the main stem to the downstream limit of the study.
- f. Scenario 7 provides control of the 2-, 10-, and 100-year frequency discharges to existing conditions from the proposed facilities to the watershed outfall and hydraulically outperforms the other scenarios.
- g. Scenario 5 provides more control for the 2-year frequency than Scenario 1. The control is approximately equal for the 10- and 100-year frequencies. However, Scenario 1 protects an additional reach of about one-half mile of Cabin Branch as compared to Scenario 5.
- h. Scenario 10 provides control to a point about half-way between existing conditions and predeveloped conditions for 2-, 10- and 100-year frequencies for key points downstream of key point A.
- i. Discharge control to at least the existing conditions is desirable for Cabin Branch to prevent higher velocities and erosion potential.

2. Recommendation

It is recommended that three SWM facilities be designed and constructed in the Cabin Branch watershed. The facilities would be located at sites 1, 3, and 5. Such a combination of facilities would control the 2-, 10-, and 100-year discharges to a point about half-way between existing and predeveloped conditions for all key points downstream of key point A.

If for any reason only one SWM facility can be constructed in Cabin Branch, the facility should be located at site 1.

E. Epilog

The development process is dynamic and the implementation of a comprehensive stormwater management system must be flexible. The Cabin Branch study recommended construction of three regional facilities and one regional facility was being designed while the study was being conducted. To date two regional SWM facilities have been constructed (10A & Site 3), Site 5 is about to be designed as a County CIP project and Site 1 is being planned as on-site controls.

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

STORM SEWER OPTIMUM DESIGN

DONG HOANG M.Sc, P.E.

INTRODUCTION.

One of the main challenge the engineer is facing is to insure that the design is technically sound and competitive in terms of costs. In other words, the optimization of a design is one of his major concerns. The optimization is becoming urgent and critical in the today world where the resources are scarce.

This paper presents the highlights of a computer model called the STORM SEWER OPTIMUM DESIGN whose objectives are to provide the least expensive design.

To evaluate the water runoff for storm sewer design the rational formula ($Q=CiA$) was the most often used.

In order to use the formula a set of rainfall intensity-duration-frequency (IDF) curves relating the rainfall intensity to the time and return period should be developed.

The IDF curves are constructed based on the extreme values of the partial duration series and the rainfall record length, consequently they yield conservative values, in term of peak discharge rate, when compared to the actual storms.

The IDF curves shape corresponds to the immediate peak situation which unlikely happens in reality. The shape of the IDF curves has also the effect of reducing the flow. As matter of fact, more water is absorbed by the infiltration when the overland water is flowing at peak over a soil whose infiltration potential is relatively high at the beginning of the rain.

Besides, the runoff coefficient, C , is too simplistic to reflect a very complicated situation.

In general, the method of evaluating the overland flow with the rational formula is too simple to reasonably represent the real situation.

Finally, the design process, including the cost computation, is laborious to the extent that it discourages the effort to explore all the situations in order to arrive to the optimum design.

Considering all the above facts, the ideal design method should:

1. be capable of making use of the real rainfall instead of evaluating it from the IDF curves
2. combine the engineering and economic aspects simultaneously
3. use method of computation which results in realistic estimation of the overland flow and the pipe flow
4. be capable of exploring all the situations quickly to arrive to the optimum design.

The STORM SEWER DESIGN OPTIMIZATION is developed according to the four points exposed above.

The model is composed of 2 parts. 1. PART ONE : HYDROGRAPH COMPUTATION 2. PART TWO : DESIGN.

PART ONE : HYDROGRAPH COMPUTATION

Once the sewer system is laid out, it is necessary to compute the runoff from each subcatchment tributary to the corresponding sewer line. The runoff is computed for each rainfall increment in the form of hydrograph based on the infiltration, the imperviousness condition, the ground slope, the Manning's coefficient of the subcatchment and the shape, slope, length, Manning's coefficient of the gutter conveying the runoff to the manhole.

In the case of designing the extension of an existing sewer system the model computes the combined hydrographs which are generated by the combination of the hydrographs of the upstream basins and the hydrographs of the area whose sewer system is to be designed.

Another task performed by the model is to create the node and line numbers system.

PART TWO : DESIGN

Using the hydrographs just created, the second part of the model will compute the design-flows.

Given the ground profile, the minimum cover, the maximum invert elevations, the underground constraints, the minimum pipe size & slope, the sewer construction elements prices, the design objectives are to find out the optimum slope, the pipe size, the pipe class & bedding types corresponding to the least sewer cost to carry the design flow. The model makes sure that the design flow at the current time step is the greatest design flow.

The design process of a line consists of varying the slope by a constant amount corresponding to the fixed drop increment a number of times consistent with the drop available. The drop available is the difference between the downstream invert elevation of the line (if a minimum slope is passed through its upstream invert) and its maximum allowable downstream invert elevation.

For each slope a diameter is calculated to carry the design flow. Corresponding to the computed diameter, two immediate surrounding commercial pipe diameters are selected. With each commercial pipe selected recompute the slope to carry the design flow. This slope should create the downstream invert elevation falling in between two values: one is the upper limit (if specified), the other is the maximum allowable downstream invert elevation. If this condition is not met, using the slope which makes the downstream invert elevation match with the maximum allowable one. Under either circumstances, once the commercial pipe diameter and the corresponding slope to carry the design flow are found, without conflicting the underground constraints, the construction cost of the line is computed. By comparison the cost corresponding to two commercial pipe diameters, the selection of the commercial pipe diameter corresponding to the least cost can be made.

The same process is repeated with a different slope which is equal to the previous slope plus the slope increment corresponding to the drop increment. The least cost between the previous and the actual and the associated elements are saved. After varying the slopes a number of times, the optimum slope and diameter can be found to carry the design flow without conflicting the underground constraints.

The model use the Manning formula for design. For a time step, the model computes the flow out and the flow stored in the line by solving for the depth such that the change in the storage is equal to the difference in the flow in and flow out. The Newton-Raphson technique is again used to find the water depth and flow.

The model computes the transition width, the earth load, the traffic load, if any, to figure out the type of pipe and the type of bedding. Complying with the design procedure in force, and the unit prices list, the model computes the sewer construction cost.

Once the design is done, it is advisable to check the behaviour of the system using the SWMM model, or any similar model with the same storm used for design. Flows adjustment factors are provided to adjust the design flows and consequently, the slope and pipe diameter, in any desired proportion to the maximum flows generated by the checking model (SWMM or any similar models).

MODEL APPLICATION

The basin served by the sewer system to be designed covers an area of 89.72 Ac with the imperviousness factor equal to 39%, and the average ground slope of 0.015. The Manning's factor, surface storage and Horton' coefficients are as follows:

Manning factor		Surface storage		Horton coefficients			
Imper- vious area	Per- vious area	Imper- vious area IN	Per- vious area IN	Maximum rate IN/HR	Infiltration		
					Minimum rate IN/HR	Decay rate 1/SEC	Maximum allowed IN
.013	.025	.062	184	3.00	.52	.00115	12.00

The total length of the system is 6700 Ft. Other informations concerning the sewer system can be found in the output listings.* The rainfall used for design is from the 10-year recurrence IDF curves of the City of Portland. Rainfall intensities are recorded for every 5-minutes. Only the first 10 rainfall steps are used for design. The rainfall is labelled: 10IDF6-00-5. It stands for 10 year recurrence storm, 6 hours duration, immediate peak & 5 minutes interval.

For comparision purpose (with the rational method) synthetic rainfall is used instead of real rainfall. The output listings are self explanatory.

THE OPTIMUM DESIGN METHOD VERSUS THE RATIONAL METHOD

To avoid spending time to find the slopes to be used in the storm sewer design with the Rational Method, the optimum slopes system created by the Optimum Design Method is used.

*See Figure 1, Tables 1-3.

By doing this, a significant amount of time is saved from the design time with the Rational Method.

In reality, it is impossible that the optimum slopes system can be found in the practice of the Rational Method because the design process, including the cost computation, is laborious to the extent that it discourages the effort of exhausting all the situation to reach the optimum design.

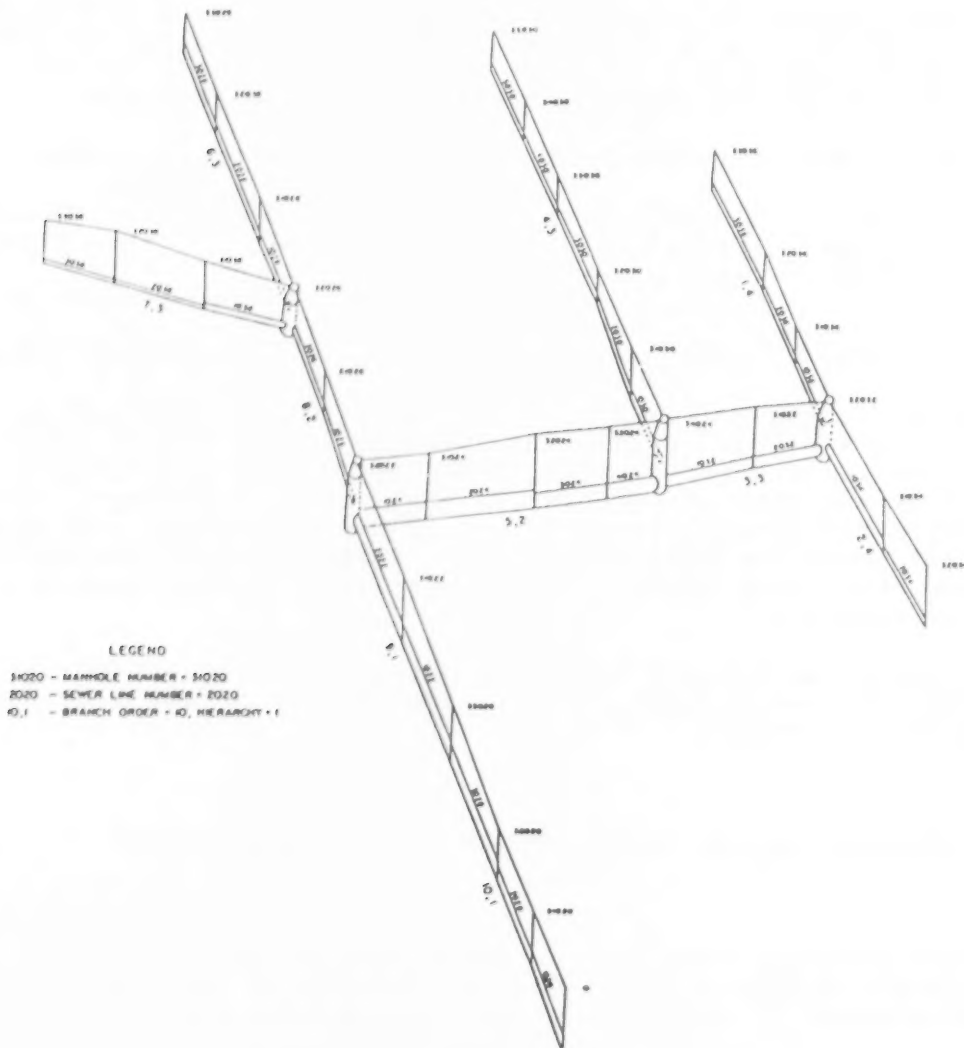


Figure 1. Storm Sewer System

Table 1. Sewer System Characteristics and Sewer Cost

LINE NUMBER	PIPE DIAMETER (IN)	PIPE SLOPE (FT/FT)	INVERT ELEVATION (FT)	PIPE LENGTH (FT)	TRENCH WIDTH (FT)	DESIGN FLOW (CFS)	VELOCITY FLOW-FULL (FPS)	PIPE CLASS	TYPE OF BEDDING	SEWER COST (\$)
3036	10	0.00264	148.21	300.00	2.5	1.13	2.06	2	C	7656
2036	12	0.00933	146.34	200.00	2.5	3.44	4.38	11	B	6424
1036	15	0.00391	145.16	200.00	2.5	4.97	4.05	11	B	6883
BRANCH NUMBER 36					COST =		20963			
2034	12	0.00200	144.30	250.00	2.5	1.39	2.03	111	B	9113
1034	15	0.00409	143.07	350.00	2.5	4.13	3.37	111	B	12987
BRANCH NUMBER 34					COST =		22100			
2032	18	0.00817	141.43	200.00	2.9	9.50	5.37	111	B	9255
1032	18	0.01271	138.26	250.00	2.9	11.84	6.70	IV	B	11839
BRANCH NUMBER 32					COST =		21094			
5030	12	0.00336	149.93	200.00	2.5	2.61	3.32	11	B	6505
4030	15	0.00572	148.78	200.00	2.5	4.89	3.98	11	B	6856
3030	15	0.01101	146.03	250.00	2.5	6.78	3.52	11	B	8371
2030	15	0.01927	142.18	200.00	2.5	8.97	7.31	111	B	7181
1030	18	0.00950	140.28	200.00	2.9	10.24	5.79	111	B	9110
BRANCH NUMBER 30					COST =		38023			
4024	24	0.00872	136.95	150.00	3.5	21.13	6.72	IV	B	10043
3024	27	0.00678	135.59	200.00	3.8	25.50	6.41	IV	B	15066
2024	30	0.00339	133.97	300.00	4.1	30.12	6.13	IV	B	24623
1024	30	0.00633	132.71	200.00	4.1	32.63	6.64	IV	B	16339
BRANCH NUMBER 24					COST =		66270			
3028	10	0.01480	151.04	200.00	2.5	2.67	4.89	2	C	5368
2028	12	0.01972	146.11	250.00	2.5	5.00	6.37	111	B	8157
1028	15	0.01352	143.41	200.00	2.5	7.51	6.12	111	B	7271
BRANCH NUMBER 28					COST =		20796			
3038	8	0.01482	149.04	200.00	2.5	1.47	4.21	2	C	5305
2038	10	0.01424	145.48	250.00	2.5	2.61	4.79	2	C	7302
1038	12	0.01136	142.64	250.00	2.5	3.80	4.83	111	B	8407
BRANCH NUMBER 38					COST =		21014			
2026	18	0.01488	139.66	200.00	2.9	12.81	7.25	111	B	8318
1026	18	0.02786	134.09	200.00	2.9	17.53	9.92	111	B	8913
BRANCH NUMBER 26					COST =		17233			
2022	30	0.01339	128.69	300.00	4.1	47.46	9.67	IV	B	24315
1022	36	0.00776	126.36	300.00	4.7	58.75	8.31	IV	B	28976
BRANCH NUMBER 22					COST =		53491			
3020	30	0.01375	122.24	300.00	4.1	48.09	9.79	IV	B	23487
2020	30	0.01572	119.09	200.00	4.1	51.43	10.48	IV	B	16078
1020	30	0.01700	115.69	200.00	4.1	53.48	10.89	IV	B	16316
BRANCH NUMBER 20					COST =		56081			
SEWER SYSTEM TOTAL COST =							337064			

Table 2. Elemental Quantity of the Sewer System

DIAMETER (IN)	LENGTH					
	CLASS 2 (FT)	CLASS 3 (FT)	CLASS II (FT)	CLASS III (FT)	CLASS IV (FT)	CLASS V (FT)
8	200	0	0	0	0	0
10	750	0	0	0	0	0
12	0	0	400	750	0	0
15	0	0	650	750	0	0
18	0	0	0	800	250	0
24	0	0	0	0	150	0
27	0	0	0	0	200	0
30	0	0	0	0	1500	0
36	0	0	0	0	300	0

PAVEMENT YD2	PAVEMENT BASE YD3	EXCAVATION YD3	BACKFILL GRAVEL YD3	BEDDING B-C YD3	FOUNDATION A STABILIZATION YD3	MANHOLE 3FT-SECTN EA
3043	676	29452	27448	380	0	1149
						191

The C-factor of the Rational Formula associated with the combined area is a weighted factor between 2 constituent areas (right & left) with 2 associated imperviousness factors. Example: the area served by manhole 3036 is equal to 1.52 Ac = 83 + .69
The C-factor of area 1.52 Ac is:

$$C = \frac{.83 \times .35 + .69 \times .40}{1.52} = .37$$

For comparison purpose, the sewer system characteristics designed by the Rational Method and the Optimum Design Method are checked by the SWMM model with the same rainfall 10IDF6-00-5 used for design.

CONCLUSION

1. The Optimum Design Method, based on the principle of the least sewer construction cost, produces the optimum sewer system characteristics which is never realized in the practice of storm sewer design with the Rational Method
2. The Optimum Design Method can make the sewer system fully used. Meanwhile, even using the optimum slopes system, the Rational Method yields a sewer system which is over designed about 67% (average percentage of flow occupancy is 33%)

Table 3. Comparison

FROM NODE NUMBER TO NODE NUMBER	LINE NUMBER	PIPE DIAMETER (IN)		PERCENTAGE OF ACTUAL DEPTH TO MAXIMUM DEPTH USING SWMM AS CHECKING MODEL.	
		Q=CiA	SSOD	Q=CiA	SSOD
Branch # 36					
33036-32036	3036	12	10	42%	100%
32036-31036	2036	14	12	43	86
31036-32032	1036	18	15	43	84
Branch # 34					
32034-31034	2034	14	12	38	*-1.61/10.00
31034-32032	1034	18	15	41	-1.55/ 8.00
Branch # 32					
32032-31032	2032	21	18	32	-1.73/ 7.93
31032-34024	1032	20	18	27	-1.69/ 8.57
Branch # 30					
35030-34030	5030	14	12	44	-0.44/ 6.00
34030-33030	4030	18	15	43	-0.43/ 6.07
33030-32030	3030	18	15	41	-0.69/ 5.22
32030-31030	2030	18	15	41	-0.88/ 5.97
31030-34024	1030	21	18	31	-0.33/ 7.82
Branch # 24					
34024-33024	4024	30	24	33	-1.51/ 9.74
33024-32024	3024	30	27	24	-0.84/10.05
32024-31024	2024	33	30	28	-0.57/10.41
31024-32022	1024	33	30	25	-0.79/ 9.03
Branch # 28					
33028-32028	3028	12	10	49	76%
32028-31028	2028	14	12	44	82
31028-32026	1028	18	15	42	86
Branch # 38					
33038-32038	3038	10	8	51	-1.50/ 6.00
32038-31038	2038	12	10	42	-2.06/ 7.46
31038-32026	1038	14	12	27	-1.80/ 7.02
Branch # 26					
32026-31026	2026	20	18	20	-0.52/ 6.36
31026-32022	1026	20	18	34	81%
Branch # 22					
32022-31022	2022	33	30	21	-0.90/ 9.29
31022-33020	1022	36	36	12	-0.27/ 9.81
Branch # 20					
33020-32020	3020	33	30	14	-1.36/ 8.14
32020-31020	2020	33	30	18	-0.54/ 7.76
31020-0	1020	33	30	19	-0.06/ 8.41

*To effectively use the sewer system, the Optimum Design Method makes the sewer surcharge (minus sign) by using the flow adjustment factors provided for each branch. The denominator is the manhole depth. Both numerator & denominator are in feet

Q=CiA stands for the Rational Method ; SSOD stands for the Optimum Design Method.

3. The Optimum Design Method can use the real rainfall for design
4. The Optimum Design Method which calculates the costs of the sewer lines, sewer branches and the sewer system as a whole makes the Cost-Benefit Analysis of alternative routes easy and effective
5. The Optimum Design Method Which computes and lists the elemental quantities regarding the sewer construction makes the preparation work for bidding accurate and fast
6. The Optimum Design Method is a very effective tool for planning as well
7. The Optimum Design Method with 2 incorporated features :
 1. the automatic way to number manholes and sewer lines
 2. the capability to design the extension of an existing sewer system, makes the preparation work less extensive and the storm sewer design more flexible
8. The Optimum Design Method is very fast and effective. It can be handled by people whose engineering knowledge is not necessarily as high as the knowledge of the people who use the Rational Method for storm sewer design.

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

ATTENDEES
USERS GROUP MEETING
January 27-28, 1983

Thomas Barnwell, Jr.
EPA
Athens, GA

Lars Bengtsson
McMaster University
Hamilton, Ontario

Vincent Berg
Montgomery County
Rockville, MD

Patrick Beron
Ecole Polytechnique
Montreal, Quebec

John Capece
University of Florida
Gainesville

Ivan Chou
ESE, Inc.
Gainesville, FL

John M. Crouse
Greenhorne & O'Mara
Riverdale, MD

Robert Dickinson
University of Florida
Gainesville

Hoang Dong
City of Portland
Portland, OR

Debbie Dunnam
University of Florida
Gainesville

David Dwornik
University of Florida
Gainesville

Kenneth G. Eggert
Simons, Li & Associates
Fort Collins, CO

Raymond A. Ferrara
Princeton University
Princeton, NJ

David Firmage
Colby College
Waterville, ME

Efi Foufoula
University of Florida
Gainesville

Yvonne Froscher
Miller & Miller
Orlando, FL

Robert Frost
University of Florida
Gainesville

Gary Goforth
Espey, Huston and Associates
Austin, TX

Philip Gronstal
Dallas Water Utilities
Dallas, TX

Brendon M. Harley
Camp, Dresser & McKee
Waltham, MA

Bruce W. Harrington
Maryland Water Resources
Administration, Annapolis, MD

James P. Heaney
University of Florida
Gainesville

Michael Helfrick Montgomery County Environmental Protection, Rockville, MD	Cheri Porter Miller & Miller Orlando, FL
Wayne C. Huber University of Florida Gainesville	Mark Robinson McMaster University Hamilton, Ontario
Maine Hutchison Oklahoma Water Resources Board	Ian Simmers Free University Amsterdam
William James McMaster University Hamilton, Ontario	Charles Simon Nat'l Council for Air & Stream Improvement, Gainesville, FL
Peter Jaffe Princeton University Princeton, NJ	Gary Trott Wiedeman & Singleton Engineers Atlanta, GA
Terrie Lee University of Florida Gainesville	William F. Walker Environmental Engineer Concord, MA
Khlifa Maalel University of Florida Gainesville	Flora Wang Louisiana State University Baton Rouge, LA
Stanley I. Mast Howard, Needles, Tammen and Bergendoff, New York, NY	Roger K. Wells HMM Associates Raleigh, NC
Ed McBean University of Waterloo Waterloo, Ontario	Stanley Wong Maryland Water Resources Administration, Annapolis, MD
Janusz Niemczynowicz Lund Institute of Technology Lund, Sweden	Grace Wood Dames & Moore Washington, DC
Alexander Padva Environmental Quality Lab Port Charlotte, FL	Jy S. Wu University of North Carolina Charlotte, NC